PILEAN: An Expert System for Pile Foundation Design

A Thesis Submitted
in Partial Fulfillment of the Requirements
for the Degree of

MASTER OF TECHNOLOGY

by

Iyer Anantharam N.

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR

May, 1995

Certificate

This is to certify that the work contained in this thesis, entitled **PILEAN - An Expert System For Pile Foundation Design**, has been carried out by *Iyer Anantharam N.* under my supervision and that this work has not been submitted elsewhere for a degree.

5/5/95

May 05, 1995
Kanpur, India

Dr. N.S.V. Kameswara Rao
Professor
Department of Civil Engineering
Indian Institute of Technology, Kanpur

Acknowledgements

It gives me immense pleasure in expressing my deep sense of gratitude and sincere thanks to Dr. N.S.V. Kameswara Rao, my thesis supervisor, who has always been a constant source of encouragement and a true guiding light.

As for the people who I have to thank for helping me at all times and making my stay at IIT, Kanpur such a pleasant experience there is no end. Let it be known that no one who deserves thanks has been forgotten and I will suffice it to say that I am highly indebted to one and all for making this M.Tech. experience a truly learning experience for me.

ANANTHARAM

Abstract

An expert system (PILEAN) has been developed in this thesis for the design of pile foundations. The system takes the results of laboratory tests on the soil, the loads and specific choice in the pile selection, if any, as input. It computes the axial and lateral capacity of the pile sections and selects a suitable pile section, pile group configuration and performs necessary checks on the selected pile group.

For the analysis, the system uses a judicious mixture of, tried and tested, empirical results and analytical methods, with aid to numerical methods wherever essential. It is hoped that the system of analysis utilized in this expert system will prove a small step forward towards bringing the field practice at par with the present state of research in pile foundations.

Although the expert system has been developed on the UNIX it can be very easily adapted to DOS with a very few minor changes and hence it can be very easily put to use.

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Chapter 1

Introduction

1.1 Piles as foundation

All loads that come on a structure have finally to be transferred to the underlying soil on which the structure is erected. This function of transfer of load from the superstructure to the soil is accomplished by the foundation of a structure.
Where the superstructure loads are less and/or when the load-bearing capacity of the upper layer of the soil is comparatively more a shallow foundation serves the purpose.
Pile foundations are generally used when the underlying soil is incapable of taking the superstructure loads or when the superstructure loads so high that they may cause unacceptable settlements of the foundation. The design of pile foundation till today remains highly empirical in nature with greater dependence on hand-rules developed from experience than from more scientific analytical methods. This in some instances is due to the highly complicated nature of analysis which can be resolved by the use of computers which are soon becoming an integral part of our everyday lives. In the expert system (PILEAN) developed

developed in this thesis an effort has been made to use a mixture of empirical and analytical methods, empirical where they have been tried and tested and are known to give good results and analytical otherwise. It is hoped that this thesis will prove a small step ahead in bringing the state of technology in the field at par with the present state of knowledge in the field of pile design.

Piles are columnar elements in a foundation which have the function of transferring load from the superstructure through weak compressible strata, onto stiffer or more compact and less compressible soils or onto rock. They may be required to carry uplift loads when used to support tall structures subjected to overturning forces from winds or waves. Piles used in marine structures are subjected to lateral loads from the impact of berthing of ships and from waves. Combinations of vertical and horizontal loads are carried where piles are used to support retaining walls, bridge piers and abutments, and machinery foundations.

1.1 Types of piles

Piles can be classified either on their material or their method of installation.

- There are three materials used for applications as pile foundations,

  Timber was the earliest used material for the obvious reasons of its availability. Its carrying capacity is limited by its girth. It can support only modest loads of upto $500kN$ and can be used to lengths of $12m$. Timber is susceptible to decay and hence the piles have to be treated before use. Also the pile can be damaged due to overdriving resulting in the *brooming* of the pile head, cracking of the shaft and unseen damage to the pile tip.

Due to the lack of reliability of timber piles, reinforced concrete piles became more popular in the late nineteenth and twentieth century. It could be cast to various desired sizes and shapes and it had the advantage of being able to carry higher loads and resist the deteriorating influences of water and soil.

Steel is being used to an increasing extent for piling due to its ease of fabrication and handling and its ability to withstand high driving stresses.

Composite pile sections can also be formed by jointing a timber section with a precast concrete section, or a precast concrete section with a steel H-section at its lower end.

- On the basis of their method of installation piles are classified as displacement piles or non-displacement piles.

The lowering of a prefabricated pile with the displacement of the surrounding soil is described as *driven precast.* It can be done by driving, vibrating, pressing or screwing.

The lowering of a prefabricated pile without displacement of the surrounding soil is described as *bored precast.* It can be done by excavating a hole and installing a pile inside, or jetting out a hole beneath a pile.

Producing a hole by driving -with or without casing- with displacement of the penetrated soil and filling it with concrete with or without reinforcement is described as *driven cast-in-situ.*

Excavating a hole, with or without casing, and filling it with concrete is described as *bored cast-in-situ.*

Piles can be obtained in a wide variety of cross-sectional shapes. Timber piles are generally either square or circular in section. Concrete piles are used in circular, square, octagonal or hexagonal sections. Steel piles can be plain tubes, box-sections, H-sections, and tapered and fluted.

1.3 Design of pile foundations

Piles when used as a foundation are generally used in groups. The general criteria for the design of pile groups is that

1. the load carrying capacity of a single pile should not be exceeded by the loads comimg on the individual pile elements,

2. the load carrying capacity of the group should not be exceeded by the total superposed loads,

3. the settlements, both total and differential, should not be such that it may result in structural damage and/or functional distress.

The load bearing capacity of a pile is dependent mainly on the properties of the soil in which it is installed and to a smaller extent on the material, type and method of installation of the pile. The calculation of the load bearing capacity is a complex matter which is presently based partly on theoretical concepts derived from the science of soil mechanics, but mainly on empirical methods based on

experience. The uncertainities present in the prediction of the bearing capacities of pile foundations are highly subjective, as the disturbance of the soil caused in the installation of the pile is likely to cause changes in the properties of the soil and also influence the soil-pile interaction behaviour.

The code of practice for design and construction of pile foundation [IS:2911 Part 1 - 1984], reflects the uncertainities in predicting the allowable or ultimate load. The code, on the point of calculation of axial bearing capacity, states, "By using static formula, the estimated value of ultimate load capacity of a typical pile is obtained, *the accuracy being dependent on the reliability of the formula and the reliability of the available soil properties for various strata*". On a later point concerning the bearing capacity of pile groups it states, "In order to determine the bearing capacities of a group of piles, a number of efficiency equations are in use. *However, it is very difficult to establish the accuracy of these efficiency equations as the behaviour of pile group is dependent on many complex factors. It is desirable to consider each case separately on its own merit*".

In the light of such uncertainities it is not only essential to correctly choose a method of analysis but also make sure, at certain stages that no unreasonable values are obtained. An attempt has been made to embody this idea in the expert system developed in this thesis.

1.4 What is an Expert System ?

An expert system is an 'intelligent', interactive computer program that can play the role of a human expert by using heuristic knowledge or rules of the thumb [Adeli (1988)].

Expert systems are a class of programs that excel in domains where judgement, expertise and rules-of-thumb are the predominant part of of the knowledge used in solving problems.

Particularly in a field like Geotechnical Engineering where *expert knowledge* counts a great deal an expert system can be a very useful tool. An expert over the years of working experience can make educated guesses, recognize impossible solutions and narrow down the field of possible solutions in a very short time. An expert system attempts to imitate this technique of the expert. The knowledge required for an expert system is obtained either from human domain experts or documented material developed by them.

Successful application of expert systems to Geotechnical Engineering problems have been made in the past. A system for the design of shallow foundations for given soil and load conditions, GEOTECH has been developed [Parikh (1989)]. An expert system for the estimation of the liquefaction potential of the ground LIQUEFY has been described by Chouicha (1994).

Expert systems however suffer from some limitations such as th following,

1. An expert system lacks the human ability to learn from experiences.

2. It lacks common sense and intuition.

3. Their performance degenerates fast near the boundaries of their expertise, *for example* if we were to define a sandy soil as one that has a cohesion value of 0 kN/m^2, any soil that has a cohesion value higher than zero, however small, will be considered as a clay. This can sometimes end up giving erroneous results.

4. Any expert system can be only as good as the heuristics used in the system.

1.5 Scope of the present work

In the expert system (PILEAN) developed in this thesis, an attempt has been made to use a judicious mixture of empirical, analytical and numerical methods. As input it takes the loads and moments coming on the structure, which can be in any of the three directions, x,y, or z and the properties of the soil in a layered profile. It computes first the axial and lateral capacities of a single pile. On the basis of the single pile capacities it computes the number of piles required in a group. It then attempts to find a suitable configuration for the group and calculates the group capacity and settlements. The pile sizes and shapes can either be user defined or from the existing database. It also optimizes the cost and selects the most economic pile section if more than one have been considered. The conceptual flowchart of the system is as shown in *figure 1.1.* Generally expert systems are developed in Artificial Intelligence (AI) languages such as PROLOG or LISP rather than in conventional procedural languages such as FORTRAN, PASCAL or C. Although the AI languages are very powerful for a database oriented system they fall short if there is a demand

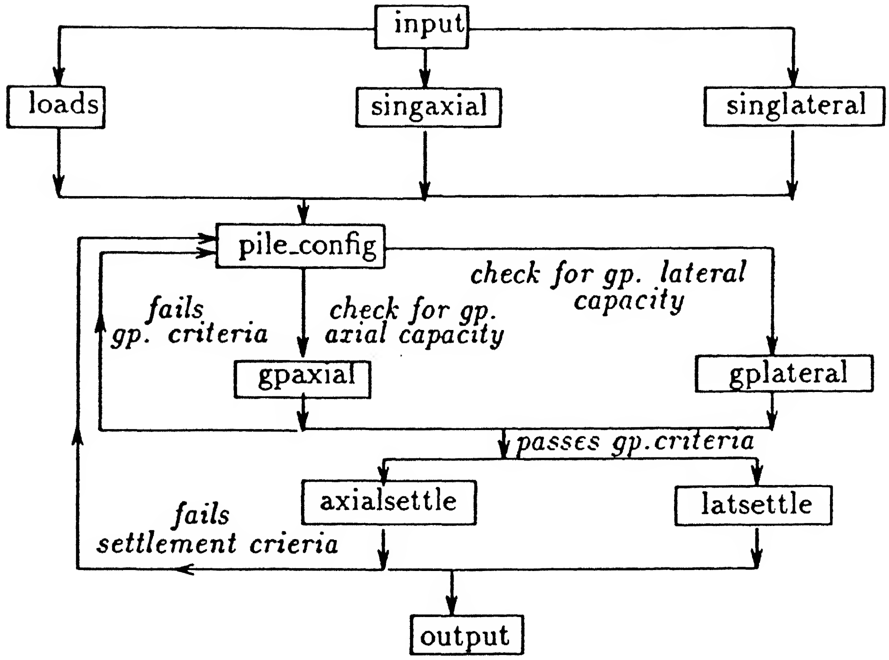


Figure 1.1: Conceptual Flowchart of the Expert System PILEAN

for intensive computation. Unlike conventional expert systems PILEAN is much less database dependent and relies more on equations. Hence C was chosen as the programming language. The system is interactive and all the inputs can be given at the time of execution. However one could also opt to make suitable changes in the input files and disable the interactive mode.

As an output it gives the diameter and shape of the selected pile, the number of piles required, the longitudinal reinforcement of the pile section, the loads coming on individual pile elements and their settlements.

1.6 Structure of the report

Since the subject matter is very vast, this thesis has been divided into parts on the basis of the subject matter. Hence Chapter 2 deals with the calculation of axial capacities of piles, Chapter 3 the calculation of lateral capacities, Chapter 4 with the computation of settlements. These three chapters have a brief survey of the relevant literature before proceeding to the methods applied. Chapter 5 deals with the decision making involved in the selection of pile configuration and the selection of the appropriate pile section and Chapter 6 with the inputs required and the outputs obtained from the system. Chapter 7 is devoted to the conclusions drawn from the present work and the improvements possible in the work as also the scope for further similar work.

Chapter 2

Axial Capacity of Piles

2.1 Single Pile

The ultimate bearing capacity of a single pile may be calculated by any of the following methods,

1. By the use of static bearing capacity equations.
2. By the use of values of Standard Penetration Test (SPT) and Cone Penetration test (CPT) results.
3. By field load tests.
4. By dynamic methods - from the analysis of pile-driving data.

For both cohesive and cohesionless soils, more than 50% of the practitioners prefer static load tests, local experience and simple empirical correlations to other methods such as direct correlations with insitu tests, effective stress analysis or capacity assessment based on dynamic measurements made during driving [Focht (1985)]. Poulos (1989) has made a compilation of available methods both

empirical and analytical used in the estimation of the axial capacity of a single pile.

A pile subjected to load parallel to its axis will carry the load partly by shear/friction generated along the shaft, and partly by normal stresses generated at the base of the pile/end-bearing [*fig. 2.1*].

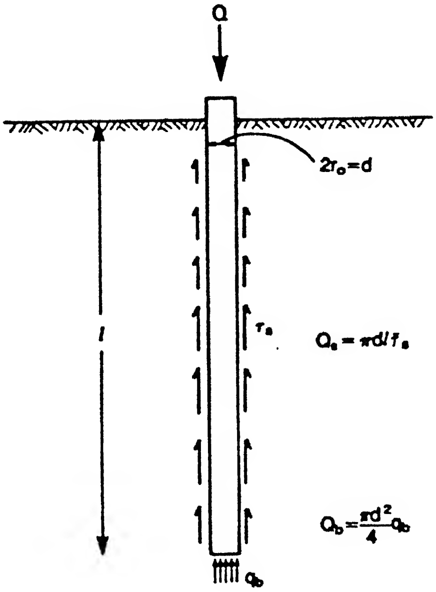


Figure 2.1: Axially Loaded Pile

The ultimate capacity, Q, of the pile under the axial load is the sum of the base capacity Q_b, and the shaft capacity Q_s. Thus

$$Q = Q_b + Q_s = A_b\, q_b + A_s\, \tau_s - W \tag{2.1}$$

where A_b = area of the pile base

q_b = end-bearing pressure

A_s = area of the pile shaft

τ_s = average limiting shear stress down the pile shaft.

W = is the weight of the pile.

Equation 2.1 can also be expressed in the form,

$$Q = Q_b + \int_0^L C(c_a + \sigma'_v K_s \tan\phi_a)dz \tag{2.2}$$

where C = the perimeter of the pile,

L = the length of the pile shaft,

c_a = the adhesion between the soil and the pile,

σ'_v = the effective vertical stress,

K_s = the coefficient of lateral pressure,

ϕ_a = angle of friction between pile and soil.

The relative magnitudes of the shaft and base capacities will depend on the geometry of the pile and the soil profile.

2.1.1 Base Capacity

The general expression for the base capacity can be given as

$$q_b = cN_c s_c d_c i_c + \sigma'_{vb} N_q s_q d_q i_q + 0.5\gamma d N_\gamma s_\gamma d_\gamma i_\gamma \tag{2.3}$$

where c_u = the undrained shear strength of the soil,

σ'_{vb} = the effective vertical stress at the level of the pile base,

γ = the unit weight of the soil,

d = diameter of the pile base,

s_c, s_q, s_γ = shape factors,

d_c, d_q, d_γ = depth factors,

i_c, i_q, i_γ = inclination of load factors.

N_c, N_q, N_γ = bearing capacity factors, which are primarily functions of the angle of internal friction, ϕ of the soil, the relative compressibility of the soil and the pile geometry.

Meyerhoff's bearing capacity factors incorporate the shape and depth effects [Bowles (1992)] and in the problem tackled in this thesis since the load is axial the inclination factor can be taken to be unity.

The expression for q_b can be simplified depending on whether the pile is embedded in a cohesive soil or a non-cohesive soil.

For a **cohesive soil** ϕ_a can be taken as zero for which $N_q = 1$ and $N_\gamma = 0$ [Poulos (1980)]. Substituting these values of the bearing capacity factors in *equation 2.3* the expression for the end bearing pressure be given as

$$q_b = N_c \, c_u + \sigma_v' N_q \tag{2.4}$$

For a pile without an enlarged base $A_b \sigma_v' \approx W$ and hence the ultimate base resistance can be expressed as

$$Q_b = A_b c_u N_c \tag{2.5}$$

Meyerhoff (1951) has shown theoretically that the bearing capacity factor, N_c, is approximately equal to 9 provided that the pile has been driven at least to a depth of 5 diameters into the bearing stratum. Randolph (1992) quotes Skempton as having stated that a linear interpolation should be made between a value of $N_c = 6$ for the case of the pile tip just reaching the bearing stratum, upto $N_c = 9$ where the pile tip penetrates the bearing stratum by 3 diameters or more. Poulos (1980) quotes a range of values suggested by different investigators.

For a **non-cohesive soil**, the term cN_c is taken as zero and the term $0.5\gamma d N_\gamma$ is neglected in comparison to the term involving N_q as being small [Poulos (1980)]. The expression in *equation 2.3* for the end bearing pressure is then modified to

$$q_b = N_q \sigma'_v \tag{2.6}$$

The term involving the weight of the pile in *equation 2.1* is again neglected [Tomlinson (1977)].

Although it may be expected that the end-bearing pressure would increase approximately proportional with depth, Randolph (1992) quotes Vesic that the end bearing pressure appears to attain a limiting value, beyond which there is no further increase observed with further penetration of the pile. Tomlinson (1977) gives this peak value of unit base resistance as 10.7 MN/m^2. Vesic has also found that the ratio of the limiting unit point to shaft resistances, q_b/q_s, of a pile at depth in a homogeneous soil-mass appears to be independent of the pile size, and is a function of the relative density of the sand and the method of installation of the pile.

Poulos (1980) has suggested a simplified distribution of the vertical stress adjacent to a pile as shown in *figure 2.2*. The effective vertical stress σ'_v is assumed to be equal to the overburden pressure upto some critical depth z_c, beyond which σ'_v remains a constant. Due to this assumption of the variation of the effective vertical stress with depth the average ultimate base resistance becomes constant beyond a particular depth.

Although values of N_q in literature vary widely, those derived by Berzantzev et al.,(1961) are widely accepted for the design of deep foundations. *Figure 2.3*

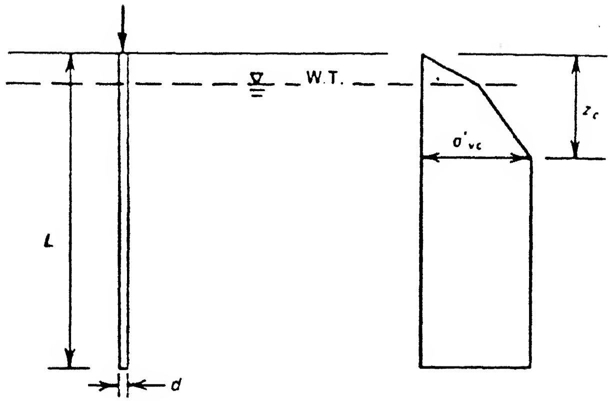


Figure 2.2: Simplified Distribution of Vertical Stress Adjacent to a Pile

shows an adapted version of the curve plotted between N_q and friction angle ϕ'.

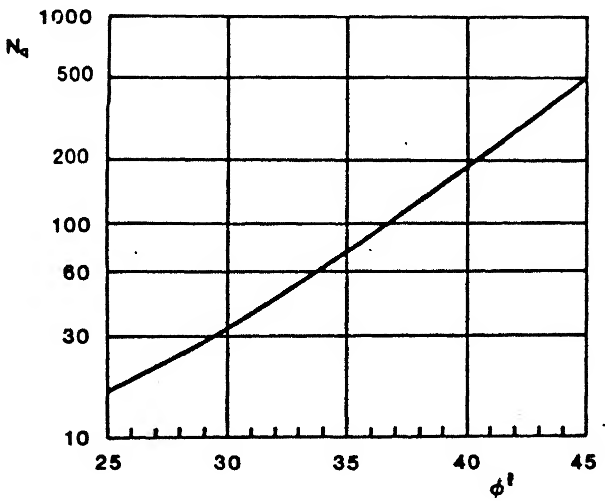


Figure 2.3: Variation of N_q with ϕ' (Berzantzev et al.(1961))

It is necessary to choose an appropriate value of ϕ' consistent with the type of non-cohesive material, its relative density, and the average stress level at failure. Following Bolton (1986), ϕ' may be related to the relative density of sand, corrected for the mean stress level, p' , and a critical state angle of friction, ϕ'_{cv}, which relates to the conditions where the soil shears at constant volume.

The corrected relative density, I_r, is given by

$$I_r \;=\; I_d\,[5.4 \;-\; \ln(p'/p_a) \;-\; 1] \tag{2.7}$$

where I_d is the uncorrected relative density, and p_a is the atmospheric pressure ($100\ kN/m^2$). Bolton suggests restricting the use of *equation 2.4* to mean effective stress levels in excess of $150\ kN/m^2$, below which the corrected relative density is taken as $I_r \;=\; 5\ I_d \;-\; 1$.

The appropriate value of ϕ' may be calculated from

$$\phi' \;=\; \phi_{cv}{}' \;+\; 3\ I_r\ degrees \tag{2.8}$$

The average mean effective stress at failure may be taken approximately as the geometric mean of the end-bearing pressure and the ambient vertical effective stress, i.e.,

$$p' \;\approx\; \sqrt{N_q}\ \sigma_v' \tag{2.9}$$

The end bearing pressure q_b, may now be calculated, for given values of ϕ'_{cv}, I_d and σ_v' by iterating between *equations 2.4* to *2.6* and the chart for N_q shown in *figure 2.3*.

For a $c-\phi$ **soil** can be calculated using Terzaghi's coeffecients N_γ, N_c and N_q in the equation

$$Q_b \;=\; B^2[1.3cN_c s_c d_c i_c \;+\; p_o(N_q \;-\; 1)s_q d_q i_q \;+\; 0.4\gamma B N_\gamma s_\gamma d_\gamma i_\gamma] \tag{2.10}$$

where N_c, N_q and N_γ are constants depending on ϕ *figure 2.4*,

c is a cohesion factor,

p_o is the effective overburden pressure at pile toe level,

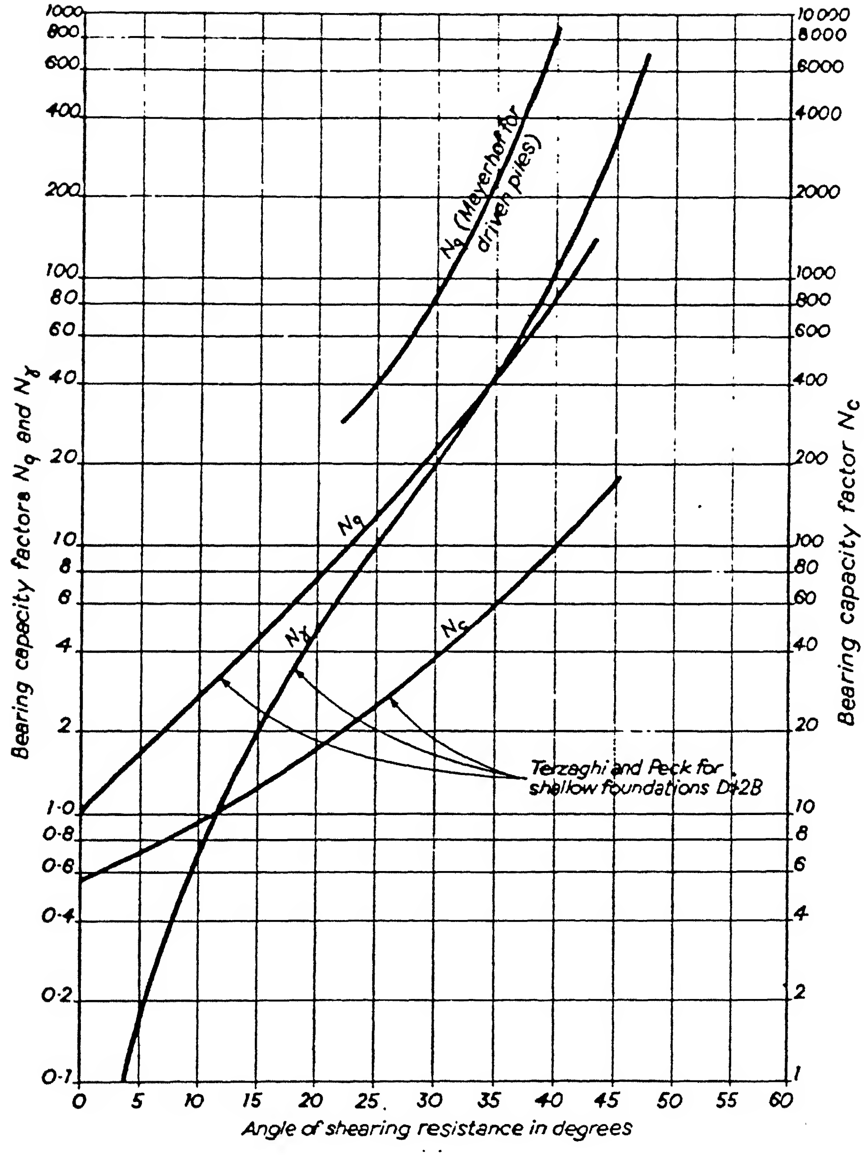


Figure 2.4: Values of N_c, N_q and N_γ (after Meyerhoff (1951) and Terzaghi and Peck (1967))

and γ is the density of the soil beneath the pile toe.

Again the shape and depth factors need not be accounted for separately as these have been accounted for in the bearing capacity factors given in *figure 2.4* and for the case of axial load the inclination factor is unity.

2.1.2 Shaft Capacity

The average limiting shear stress τ_s is given by the Coulomb's expression

$$\tau_s = c_a + \sigma_n \tan \phi_a \tag{2.11}$$

where c_a = adhesion

σ_n = normal stress between pile and soil

ϕ_a = angle of friction between pile and soil

Since piles in **cohesive soil** develop a high proportion of their overall capacity along the shaft, a considerable amount of research has gone into the development of reliable methods for the estimation of the values of their shaft capacity. Historically, the shaft capacity has been determined in terms of the undrained shear strength of the soil, by means of an adhesion factor, α [Tomlinson (1977)].

$$\tau_s = \alpha\, c_u \tag{2.12}$$

The adhesion factor depends partly on the cohesive strength of the soil and partly on the nature of the soil above the bearing startum of clay into which the piles are driven. Design curves for the adhesion factors have been presented and these are reproduced in *figure 2.5.*

Randolph et al., (1979) have opined that the appropriate value of skin friction will not only depend on the shear strength of the soil, but also on its past history

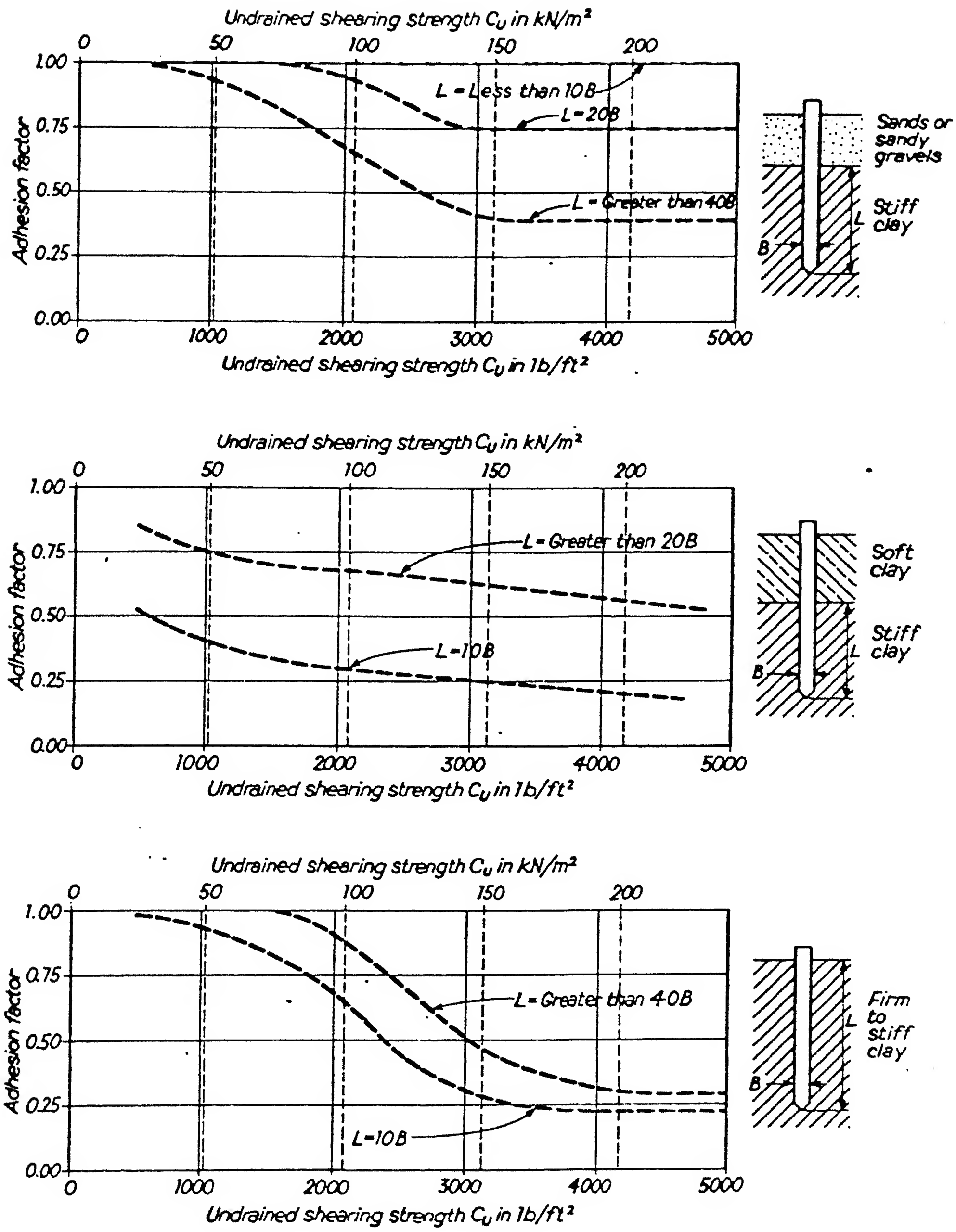


Figure 2.5: Design Curves for Adhesion Factors (Tomlinson (1978))

and overconsolidation ratio. Assuming that the value α is unity for normally consolidated clay, they have given the expressions for α as

$$\alpha = (c_u/\sigma'_v)^{0.5}_{nc} \; (c_u/\sigma'_v)^{-0.5} \quad for \; c_u/\sigma'_v \; \leq \; 1, \tag{2.13}$$

and

$$\alpha = (c_u/\sigma'_v)^{0.5}_{nc} \; (c_u/\sigma'_v)^{-0.25} \quad for \; c_u/\sigma'_v \; > \; 1. \tag{2.14}$$

The subscript nc refers to the normally consolidated state of the soil.

Chandler [Randolph (1992)], suggested an alternative method, considering the bond between pile and soil as purely frictional in nature, with the resulting skin friction a function of the normal effective stress, σ'_n and interface angle, δ, has been described. The normal stress is related to the effective overburden stress, σ'_v, by a factor, K to give

$$\tau_s = \sigma'_n \tan\delta = K \, \sigma'_v \tan\delta = \beta \, \sigma'_v \tag{2.15}$$

where $\beta = K \tan\delta$. The value of K will depend on the type of pile (driven or bored) and the past history of the soil. For piles in soft, normally consolidated or lightly overconsolidated clay, Burland and Parry and Swain [Poulos (1980)] have suggested values of K lying between $(1 - \sin\phi')$ and $\cos^2\phi'/(1 + \sin^2\phi')$. A different approach for the calculation of the ultimate shaft capacity is due to Vijayvergiya and Focht [Poulos (1980)] for steel-pipe piles. They concluded from an examination of a number of loading tests on such piles that

$$\tau_s = \lambda(\sigma'_m + 2c_m) \tag{2.16}$$

where σ'_m = mean effective vertical stress between ground surface and pile tip,

c_m = average undrained shear strength along pile,

and λ = a dimensionless coeffecient.

λ was found to be a function of pile penetration and is plotted in *figure 2.6.*

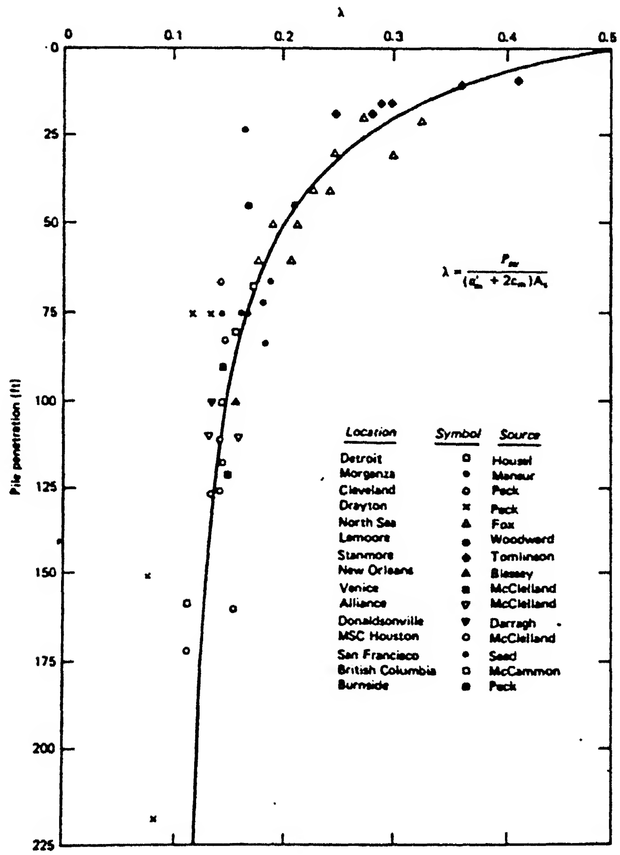


Figure 2.6: Frictional Capacity Coefficient λ vs. Pile Penetration (ft) (1ft = 0.3048m) (Vijayvergiya and Focht (1972))

The equation used to compute the shaft capacity of a pile in a **non-cohesive soil** is of the form of *equation 2.11.* Vesic has observed similar trends in the value of skin friction as for the base resistance showing a tendency towards a limiting value. Randolph (1992) suggests that the factor K in *equation 2.11* may be assumed to vary in a similar fashion to N_q and that for a full displacement driven

pile K may be estimated from

$$K \approx N_q/50 \tag{2.17}$$

and that the angle of internal friction δ may be taken equal to the critical state angle of internal friction ϕ'_{cv}. Hence

$$\tau_s = \beta\, \sigma'_v = \frac{N_q}{50}\sigma'_v\, tan\phi'_{cv} \tag{2.18}$$

The shaft capacity for a $c-\phi$ soil may be determined by considering the adhesion and friction components separately.

2.1.3 Codal Provisions

The code of practice [IS:2911 - Part 1 -1984], for the determination of the ultimate bearing capacity of a single pile, suggets the use of the static bearing capacity equation.

For a cohesionless soil it recommends the estimation of N_q from Berzantzev's curve *fig. 2.3*, the value of δ to be assumed equal to ϕ and K as between 1 and 3 for driven piles and 1 and 2 for bored piles.

For a cohesive soil the code recommends the use of $N_c = 9$ and α depending upon N value from the Standard Penetration Test (SPT).

In case that the full static penetration data are available it gives a table for the estimation of the shaft capacity dependent on the type of soil.

2.1.4 Method Adopted

- Base Capacity

used for the determination of the axial pile capacity. The assumptions made for a **cohesive** and **non-cohesive** soil are as below,

- Cohesive soils : *Equation 2.2* and N_c assumed according to Meyerhoff's suggestion as 9.

- Cohesionless soils : *Equation 2.3*, N_q determined iteratively between *equations 2.4 to 2.6* and *figure 2.3* i.e., Berzantzev's curve for N_q vs ϕ, as suggested by Bolton and as recommended by [IS:2911 - Part 1 -1984. *Figure 2.2* as suggested by Poulos to limit σ_v' is used.

• Shaft Capacity

- Cohesive soils : *Equation 2.9* is used and α is determined from the curves given by Tomlinson *figure 2.5.*

- Cohesionless soils : *Equation 2.15* is used for this purpose and N_q is again determined iteratively between *equations 2.4 to 2.6* and *figure 2.3.*

As a precaution that no unrealistic values are obtained the base resistance and shaft friction values are limited by the maximum values given in *table 2.1* and *table 2.2* [Winterkorn (1986)]. They base resistance values are also checked to the absolute maximum value of 10.7 MN/m^2 suggested by Tomlinson.

Average Depth of Layer, m	Sand, Fine Sand: Coarse to Medium	Sand, Fine Sand: Fine	Sand, Fine Sand: Rock Flour	Silts and Clay Consistency Index I_c: 0.8	0.7	0.6	0.5	0.4	0.3	Screw and Bored Piles
1	3.5	2.3	1.5	3.5	2.3	1.5	1.2	0.5	0.2	0.8
2	4.2	3.0	2.0	4.2	3.0	2.0	0.7	0.7	0.3	1.1
3	4.8	3.5	2.5	4.8	3.5	2.5	2.0	0.8	0.4	1.3
4	5.3	3.8	2.7	5.3	3.8	2.7	2.2	0.9	0.5	1.4
5	5.6	4.0	2.9	5.6	4.0	2.9	2.4	1.0	0.6	1.5
7	6.0	4.3	3.2	6.0	4.3	3.2	2.5	1.1	0.7	1.6
10	6.5	4.6	3.4	6.5	4.6	3.4	2.6	1.2	0.8	1.7
15	7.2	5.1	3.8	7.2	5.1	3.8	2.8	1.4	1.0	1.8
20	7.9	5.6	4.1	7.9	5.6	4.1	3.0	1.6	1.2	2.0
25	8.6	6.1	4.4	8.6	6.1	4.4	3.2	1.8	–	2.2
30	9.3	6.6	4.7	9.3	6.6	4.7	3.4	2.0	–	2.4
35	10.0	7.0	5.0	10.0	7.1	5.0	3.6	2.2	–	2.6

Table 2.1: Maximum Unit Values of Mantle Friction, ton/m^2

2.2 Pile Group

It is common practice to calculate the capacity of a pile group by means of an efficiency factor η given as,

$$\eta = \frac{ultimate\ load\ capacity\ of\ pile\ group}{sum\ of\ ultimate\ load\ capacities\ of\ individual\ piles} \tag{2.19}$$

Several empirical efficiency formulae are used to try to relate group efficiency to pile spacings for piles in a **cohesive soils** [Poulos(1980)]

1. Converse-Labarre formula, [Poulos(1980)]

$$\eta = 1 - \zeta[\frac{(n-1)m + (m-1)n}{mn}]/90 \tag{2.20}$$

2. Feld's rule, which reduces the calculated load capacity of each pile in a pile group by 1/16 for each adjacent pile taking no account of the pile spacing.

Depth of Pile Tip, m	Granular Soils	Gravel	Coarse Sand	–	Medium Sand	Fine Sand	Coarse Silt
	Cohesive Soils I_c	1.0	0.9	0.8	0.7	0.6	0.5
4		820	530	380	280	180	120
5		880	560	400	300	190	130
7		950	600	430	320	210	140
10		1050	680	490	350	240	150
15		1170	750	560	400	380	160
20		1250	820	620	450	310	170
25		1340	880	680	500	340	180
30		1420	940	740	550	370	190
35		1500	1000	800	600	400	200

Table 2.2: Ultimate Values of Specific Tip Resistance, ton/m^2

3. A rule in which the calculated load capacity of each pile is reduced by a proportion I for each adjacent pile where

$$I = \frac{1}{8}d/s \qquad (2.21)$$

For pile groups in sands it is fairly well established that the group efficiency may often be greater than 1.

The axial capacity of a pile group may be calculated in much the same way as that for a single pile, the only difference being that the failure of the pile group as a block has now to be considered and hence the terms A_s and A_b would now represent the block surface area and the base area of the block as shown in *figure 2.7.*

2.2.1 Base Capacity

For a **cohesive soil** the base capacity *equation 2.2* would have to be used with the difference that the bearing capacity factor N_c would now have to be taken

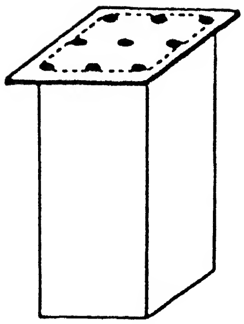


Figure 2.7: Block Failure Surface for Pile Groups

as given by Skempton [Randolph (1992)],

$$N_c = 5(1 + 0.2B/L)[1 + (l/12B)] \tag{2.22}$$

where B and L are the breadth and length of the pile group in plan and l is the embedded pile length. A limiting value of 1.5 is suggested for the correction factor $[1 + (l/12B)]$.

For a **non-cohesive soil** the term due to the self-weight of the soil below the bearing level may no longer be insignificant and the end-bearing would have to be calculated by the equation,

$$q_b = N_q \sigma_v' + 0.4\gamma B N_\gamma \tag{2.23}$$

where B is the width of the pile group (assumed aproximately square). Terzaghi's values of the bearing capacity factors N_q and N_γ are tabulated in *table 2.3*.

2.2.2 Shaft Capacity

For a **cohesive soil** the calculations remain the same as for the shaft capacity of a single pile with the change in the area of the frictional surface A_s to the surface area of the sides of the block shown in *figure 2.7*.

ϕ°	N_c	N_q	N_γ
0	5·7	1·0	0·0
5	7·3	1·6	1·5
10	9·6	2·7	1·2
15	12·9	4·4	2·5
20	17·7	7·4	5·0
25	25·1	12·7	9·7
30	37·2	22·5	19·7
35	57·8	41·4	42·4
40	95·7	81·3	100·4
45	172·3	173·3	297·5
50	347·5	415·1	1153·0

Table 2.3: Terzaghi's Bearing Capacity Factors

For a **cohesionless soil** the *equation 2.15* would now become,

$$\tau_s \;=\; K_o\sigma_v' \; tan\phi' \qquad (2.24)$$

where K_o is the coeffecient of earth pressure at rest

$= \; (1 \; - \; \sin\phi')/(1 \; + \; \sin\phi')$

In all cases the capacity of the group must be restricted to the sum of the individual pile capacities.

2.2.3 Codal Provisions

The code of practice [IS:2911 - Part 1 -1984] while questioning the accuracy of the efficiency formulae in use suggests that they be used with caution.

2.2.4 Method Adopted

Since there is a considerable variation in the η values obtained by the different efficiency formulae and Chellis (1962) after a comparison of the results obtained

by these methods with field test finds that there is little correlation between the two, the use of efficiency equations has been avoided.

Instead the static bearing capacity equations have again been utilised as outlined in *section 2.2.1* and *2.2.2*.

Chapter 3

Lateral Capacity of Piles

3.1 Introduction

Deep foundations subjected to lateral loads should be designed so that they satisfy three conditions [Duncan (1994)]

1. The pile or drilled shaft should be able to carry the imposed load with an adequate margin of safety against failure in bending.

2. The deflection of the foundation under the load should not be larger than the tolerable deflection for the structure it supports.

3. The soil around the pile or shaft should not be loaded so heavily that it reaches its ultimate load carrying capacity.

3.2 Single Pile

3.2.1 Design Philosophies

There are different design philosophies prevalent for the estmation of the lateral carrying capacity of a single pile.

- **Conventional Statical Approach :** The simplest method of estimating the ultimate lateral resistance of a floating pile is to consider the statics of the pile by approximating a soil resistance profile along the length of the pile. **Brom's theory** [Brom (1964 a and b)] is an application of this philosophy in so much that the simplifications are made to the ultimate soil resistance distribution along the pile and also full consideration is given to restrained or fixed-headed piles as well as unrestrained or free headed piles. For a general distribution of soil resistance with depth the method given by Brinch Hansen, described in detail by Tomlinson (1977) may be used.

- **Subgrade Reaction Approach :** This approach is based on the assumption that the soil reaction p is proportional to the deflection of the pile. The ratio of the soil reaction to the deflection is called the soil modulus E_s. Solutions have been developed for E_s constant with depth, [Hetenyi (1946)] and also for variation of E_s with depth and layered soil. A mention of these theories can be found in Poulos (1980).

- $p - y$ **Curves :** The Subgrade Reaction approach is only applicable to the deflection of pile within the elastic compression of soil caused by the lateral loading of piles. The $p - y$ curves developed by Reese [Tomlinson (1977)] represent the deformation of the soil at any given depth below the soil surface for a range of horizontally applied pressures from zero to the stage of yeilding of the soil in ultimate shear.

- **Charcteristic Load Method :** The Characteristic Load Method described by Duncan (1994), like the $p-y$ curve approach, takes into account the nonlinearity in the soil response to a lateral load. It gives nondimensional characteristic loads and moments as a means of normalising the soil response. Curves are then given for the estimation of the ratio of applied load to the characteristic load and applied moment to the characteristic moment for different levels of deflections.

3.2.2 Codal Provisions

The code of practice [IS:2911 - Part 1 - 1984] states that *"The lateral load carrying capacity of a single pile depends not only on the horizontal modulus of subgrade reaction of the surrounding soil but also on the structural strength of the pile shaft against bending consequent upon application of a lateral load. While considering lateral load of piles, effect of other coexistent loads including the axial load on the pile should be taken into consideration for checking the structural capacity of the shaft."* Values of modulus of horizontal subgrade reaction are given for different types of soil. Also the fixity length for the various values of the modulus of subgrade reaction are given in the form of graphs.

3.2.3 Method Adopted

In most cases consideration of bending moments and deflection govern design, because the ultimate load carrying capacity of the soil is reached only at very large deflections. The statical approach does not consider the load-deflection response.

The method of $p-y$ curves requires a great deal of time to develop the input and is highly computer intensive.

The characteristic load method works only for uniform soil conditions, i.e., it assumes that the pile is embedded in the same type of soil throughout its length. Hence the method adopted is one based on the beam on elastic foundation approach described by Vlasov et al., (1960), Kameswara Rao et al.,(1971) called the method of initial parameters. Since the method suggested by Brinch-Hansen is much more easy to apply for the case of a short pile it has been used in preference to the beam on elastic foundation approach only for the case of a short pile.

3.2.4 Long Piles - Beam on elastic foundation approach

The single parameter model for a beam on elastic foundation problem can be expressed by the differential equation [Hetenyi (1946)]

$$E_pI_p\frac{d^4v}{dx^4} + kv = q \qquad (3.1)$$

where E_p is the Young's modulus of the material of the beam,

I_p is the moment of inertia of the cross-section of the beam,

q is the load distribution on the beam,

and k is the modulus of subgrade reaction.

For the case of a beam-column the differential equation can be given by [Hetenyi(1946)]

$$E_pI_p\frac{d^4v}{dx^4} - N\frac{d^2v}{dx^2} + kv = q \qquad (3.2)$$

where N is the axial load on the beam-column.

Vesic (1961) has analysed an infinite (horizontal) beam on an elastic foundation and compared the results with those obtained by the use of Winkler's hypothesis. He concludes that the problem of bending of an infinite beam resting on a semi-infinite elastic subgrade can be treated with reasonable accuracy by the use of the concept of a coeffecient of subgrade reaction. He has given an expression for the value of k, which can be adopted for piles as

$$k_{\infty} = \frac{1.3}{B} \sqrt[12]{\frac{E_s B^4}{E_p I_p}} \frac{E_s}{1 - \nu_s^2} \tag{3.3}$$

where B is the beam width

and E_s, ν_s are elastic constants of the soil.

The homogeneous solution of *equation 3.1* is given by

$$m^4 + 4\lambda^4 = 0 \tag{3.4}$$

The roots of this equation are given by

$$m_{1,2,3,4} = \pm\lambda(1 \pm i)$$

The homogeneous solution of *equation 3.2* for a beam-column is given by

$$E_p I_p m^4 - N m^2 + k = 0 \tag{3.5}$$

with roots

$$m_{1,2,3,4} = \pm(\alpha \pm i\beta)$$

where $\alpha = \sqrt{\lambda^2 + \frac{N}{4E_p I_p}}$

and $\beta = \sqrt{\lambda^2 - \frac{N}{4E_p I_p}}$

Hence the homogeneous solution for *equation 3.1* can be written as

$$v_H = c_1 F_1 + c_2 F_2 + c_3 F_3 + c_4 F_4 = c^T F = F^T c \tag{3.6}$$

c is the matrix of constant coeffecients from the boundary conditions,

$$c = \begin{Bmatrix} c_1 \\ c_2 \\ c_3 \\ c_4 \end{Bmatrix}$$

F is the basis of solution and is given by

$$F = \begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \end{Bmatrix}$$

$$= \begin{Bmatrix} \cosh \lambda x \ \cos \lambda x \\ \cosh \lambda x \ \sin \lambda x \\ \sinh \lambda x \ \cos \lambda x \\ \sinh \lambda x \ \sin \lambda x \end{Bmatrix}$$

For the case of a beam-column the basis of solutions would be

$$\begin{Bmatrix} e^{\alpha x} \cos \beta x \\ e^{-\alpha x} \cos \beta x \\ e^{\alpha x} \sin \beta x \\ e^{-\alpha x} \sin \beta x \end{Bmatrix}$$

The differential of the basis of solutions for the homogeneous solution *equation 3.4* can be expressed as

$$\frac{d\{F\}}{dx} = [N_H]\{F\} \tag{3.7}$$

where N_H is given by

$$N_H = \begin{bmatrix} 0 & -\lambda & \lambda & 0 \\ \lambda & 0 & 0 & \lambda \\ \lambda & 0 & 0 & -\lambda \\ 0 & \lambda & \lambda & 0 \end{bmatrix}$$

The paramaters displacement v, slope v', moment M_b, and shear force V at any point can be expressed as

$$v = \{c\}^T\{F\}$$

$$
\begin{aligned}
v' &= \{c\}^T[N_H]\{F\} \\
M_b &= -E_pI\{c\}^T[N_H]^2\{F\} \\
V &= -E_pI\{c\}^T[N_H]^3\{F\}
\end{aligned}
$$

Hence

$$
\left\{ \begin{array}{c} v \\ v' \\ M_b \\ V \end{array} \right\}_{x=0} = [B]_{x=0}\{c\}
$$

If we refer to the matrix on the left of the equality as $\{I_p\}$, the matrix of initial parameters, we can write

$$
\{I_p\} = [B]_{x=0}\{c\} = [A]\{c\} \tag{3.8}
$$

Hence the $\{c\}$ matrix can be written as

$$
\{c\} = [A]^{-1}\{I_p\} = [G]\{I_p\} \tag{3.9}
$$

From *equations 3.6* and *3.7* we get

$$
\{I_p\} = [B]\{c\} = [B][G]\{I_p\} = [K]\{I_p\} \tag{3.10}
$$

The total solution can be given by

$$
v_T = v_H + v_P \tag{3.11}
$$

where v_P is the particular solution.

If the parameters at any point (*say* $x = l$) are represented as $\{F_p\}$, the matrix of final parameters, they can be written as

$$
\{F_p\} = [K]\{I_p\} - \{F_{part}\} \tag{3.12}
$$

where $\{F_{part}\}$ is the matrix of the functions depending on the external load. These can be obtained by multiplying the relevant terms of the $[K]$ matrix by the force applied.

For the case of a lateral load at the head of the pile the terms of the $\{F_{part}\}$ can be obtained as

$$\begin{aligned} F_v &= P(K_{1,4}) \\ F_{v'} &= P(K_{2,4}) \\ F_M &= P(K_{3,4}) \\ F_V &= P(K_{4,4}) \end{aligned}$$

where P is the lateral force applied at the head of the pile.

For a layered soil profile one could proceed down the length of the pile by expressing the final parameters, at the bottom of the layer, in terms of the initial parameters, at the top of the layer, and considering these final parameters as the initial parametrs for the subsequent layer. Proceeding in this fashion one could express the parameters at the bottom of the pile in terms of the parameters at the top of the pile.

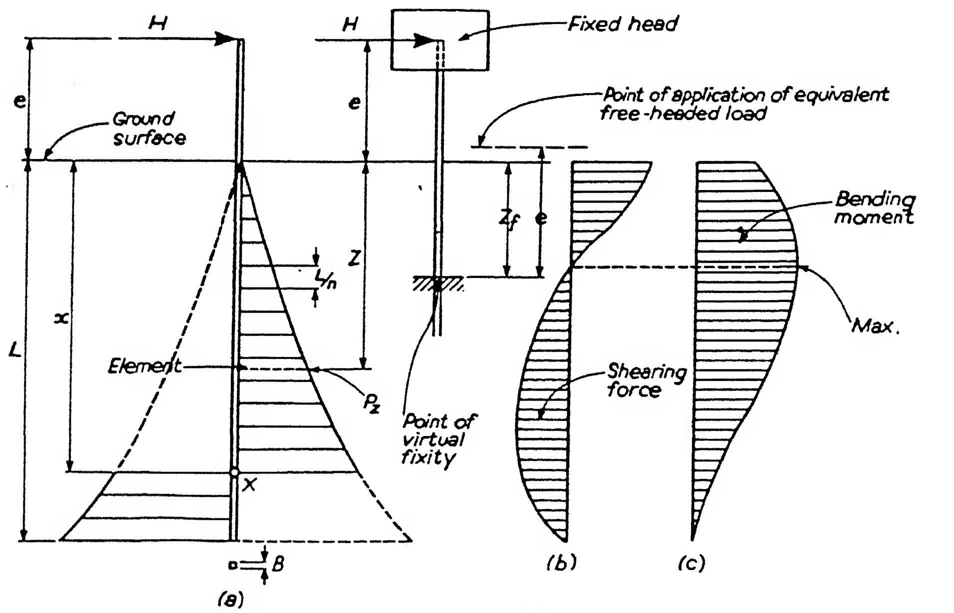
Concept of an infinitely long beam

For a pile with a lateral load applied at its head the deflection and slope, tend to zero with increasing depth as is for a beam with a concentrated load as we move away from the point of application of the load. Hetenyi (1946) has found that this distance can be expressed as $1.5\pi/\lambda$.

Hence if we were to consider a pile with length greater than $1.5\pi/\lambda$ as a infinitely long pile, we could determine the parameters at the top of the pile. Once these have been determined we could proceed from top down and calculate the lateral carrying capacity as the sum of the soil reaction over the length, kv.

3.2.5 Short Piles - Brinch Hansen's Method

The resistance of the pile to rotation about the point X in the *figure 3.1* is given by the sum of the moments of the soil resistance above and below this point. The passive resistance diagram is divided into a convenient number n of



(a) Soil reactions (b) Shearing force diagram (c) Bending moment diagram

Figure 3.1: Brinch-Hansen's Method for Calculating Ultimate Lateral Resistance of Short Piles

horizontal elements of depth L/n. The unit passive resistance of an element at depth z below the ground surface is then given by

$$p_z = p_{oz}K_{qz} + cK_{cz} \tag{3.13}$$

where p_{oz} is the effective overburden pressure at depth z, c is the cohesion of the soil at depth z, and K_{qz} and K_{cz} are the passive pressure coefficients for the frictional and cohesive components respectively at depth depth z (*fig. 3.2*).

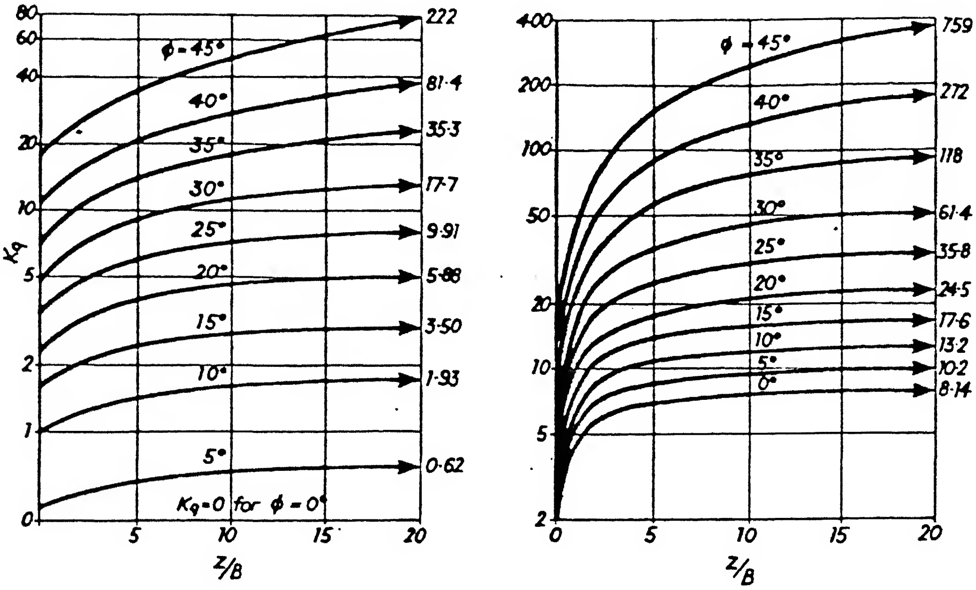


Figure 3.2: Brinch-Hansen's Coefficients K_q and K_c

The total passive resistance on each horizontal element is $p_z(L/n)B$ and by taking the moments about the point of application of the horizontal load,

$$\sum M = \sum_{z=0}^{z=x} p_z \frac{L}{n}(e + z)B - \sum_{z=x}^{z=l} p_z \frac{L}{n}(e + z)B \tag{3.14}$$

The point of rotation has to be obtained by trial and error and it is correctly obtained when $\sum M = 0$. Having obtained the center of rotation from *equation 3.12*, the ultimate lateral resistance of the pile to the horizontal force H_u can be obtained by taking moments about the point of rotation, when

$$H_u(e + z) = \sum_{0}^{x} p_z \frac{L}{n} B(x + z) + \sum_{x}^{x + L} p_z \frac{L}{n} B(z - x) \tag{3.15}$$

The ultimate bending moment, which occurs at the point of zero shear, should not exceed the ultimate moment of resistance M_u of the pile shaft. However it

is quite often found that M_u is exceeded. For cases where this does take place the ultimate lateral load carrying capacity is calculated on the basis of a scheme suggested by Carter and Kulhawy which is documented by Randolph (1992). They have presented solutions for rigid piles in homogeneous soils. The ground deflections are given by,

$$u = 0.2\frac{H}{G_c r_0}\left(\frac{l}{r_0}\right)^{-(1/3)} + 0.15\frac{M}{G_c r_0^2}\left(\frac{l}{r_0}\right)^{-(7/8)} \tag{3.16}$$

$$\theta = 0.15\frac{H}{G_c r_0^2}\left(\frac{l}{r_0}\right)^{-(7/8)} + 0.4\frac{M}{G_c r_0^3}\left(\frac{l}{r_0}\right)^{-(5/3)} \tag{3.17}$$

Again the maximum permissible deflection has to be used and assuming that the pile head is embedded in a pilecap, θ can be taken as 0.

3.2.6 Structural Checks

It has to be checked that the solution obtained does not violate either the maximum deflection criterion, the limiting moment criterion or the maximum permissible shear force criterion. The maximum permissible lateral deflection of the pile would depend on the use to which the superstructure is to be put and hence is an user-defined criterion. The maximum moment of a pile section can be calculated from the shape, dimensions and quantity of reinforcement steel in the pile section by the limit state theory. The minimum quantity of steel to be used as longitudinal reinforcement for bored piles is given as 0.4% of the cross-sectional area of the pile or as required to cater for handling stresses [IS:2911 Part 1 - Section 4 - 1984]. The minimum quantity of steel to be used as longitudinal reinforcement in a driven pile is given as [IS:2911 Part 1 - Section 3 - 1984]:

- For piles of length less than 30 times the least width - 1.25 percent,

- For piles with a length 30 to 40 times the least width - 1.5 percent,

- For piles with a length greater than 40 times the least width - 2 percent.

3.3 Pile Group

In the estimation of the lateral load capacity of a pile group an approach similar to that adopted for the calculation of vertical load capacity can be taken. The group capacity of a group of piles is the smaller of

1. n times the lateral load capacity of a single pile,

2. The lateral load capacity of an equivalent single block containing the piles in the group and the soil in between them.

3.3.1 Methods available

The concept of efficiency factor for lateral loading, like one for vertical loading, has been suggested by Poulos (1980). He has attempted to plot curves of group efficiency versus spacing/pile diameter.

Randolph (1992) has suggested avoiding calculations based on efficeincy factors and instead suggests a method taking into account the shear stresses developed on the soil between two piles as shown in *figure 3.3*.

He gives the expression

$$P_u = 2K\sigma'_v s \, tan\phi' \qquad (3.18)$$

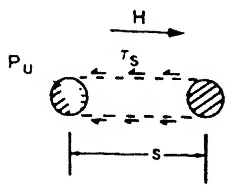


Figure 3.3: Plan View of Block Failure under Lateral Load

where K is the earth pressure coeffecient

σ'_v the effective vertical stress

and ϕ' the angle of internal friction.

However he states that the value of K to be used in the above expression is open to question.

3.3.2 Codal Provisions

The code of practice [IS:2911 - Part 1 - 1984] is silent on the topic of the design of pile groups for lateral loads.

3.3.3 Method Adopted

Since the lateral capacity of a pile group will rarely be critical in design of the group and since there is a dearth of reliable techniques for the estimation of the lateral capacity of a pile group, the capacity is calculated as the sum of the individual capacities of the piles in the group.

Chapter 4

Settlement of Piles

4.1 Introduction

The settlement of a pile is the sum of the immediate or elastic settlement and the long-term or consolidation settlement. Only the immediate or elastic settlement of the pile/pile group has been computed and hence only this topic is addressed in this chapter.

The settlement analysis of piles is a topic on which extensive research has been carried out.

4.2 Settlement under Axial Load - Single Pile

4.2.1 Methods of Analysis

The existing methods of analysis can be broadly categorized into three, Poulos[1980],

1. The **Load Transfer Method** developed by Coyle and Reese (1964), uses measurement relationships between pile resistance and pile movement at various points along the pile. In this method the pile is divided into a finite

number of segments. Assuming a tip movement, the movement of the pile segment at mid-height is estimated from load-transfer/shear strength versus pile movement curves and the elastic deformation equation in an iterative manner. Thus one proceeds up the length of the pile to obtain the load and displacement of the pile head. From a series of such calculations load-settlement curves can be plotted.

2. **Analyses based on Elastic Theory** have been carried out by many investigators [Poulos and Davies (1968); Butterfield and Banerjee (1971); Banerjee and Davies (1978) and Randolph and Wroth (1978)]. In these analyses the pile is divided into a number of uniformly loaded elements and a solution is obatined by imposing the compatibility constraints between the displacement of the soil and the pile. The displacements of the pile are obtained by considering the compressibility of the pile under axial loading and the soil displacements in most cases are obtained by using Mindlin's equations for the displacement of a soil mass caused by a loading within the mass.

3. Finite element solutions have been described by Ellison et al.(1971), Desai (1974), Lee (1973), and Balaam et al.(1975) among others.

4.2.2 Codal Provisions

The code of practice [IS:2911-1984] states that, *"The settlement of the pile obtained at safe load/working load from the load-test results on a single pile shall not be directly used for casting the settlement of a structure unless experience from similar foundations on its settlement behaviour is available. The*

average settlement may be assessed on the basis of subsoil data and loading details of the structure as a whole using the principles of soil mechanics."

4.2.3 Method Adopted

The method of analysis adopted is the one given by Randolph and Wroth (1978). This theory is based on the concept that load is transferred from the pile shaft by shear stresses generated in the soil on vertical and horizontal planes, with little change in the vertical normal stress, except near the base of the pile. Schematically, a pile may be considered as surrounded by concentric cylinders of soil, with shear stresses on each cylinder. For vertical equilibrium the magnitude of shear stresses on each cylinder must decrease inversely with the square of the surface area of the cylinder. Writing the shear stresses on the pile shaft as τ_0, the shear stresses at a radius r is given by

$$\tau = \frac{\tau_0 r_0}{r} \tag{4.7}$$

The shear strain, γ, in the soil is given by, $\gamma = \tau/G$. Since the main deformation in the soil will be vertical, the shear strain may be written approximately as

$$\gamma \approx \frac{dw}{dr} \tag{4.8}$$

where w is the vertical deflection.

These relationships may be assembled and integrated to give

$$w = \int_r^{r_m} \frac{\tau_0 r_0}{Gr} dr = \frac{\tau_0 r_0}{G} \ln(r_m/r) \tag{4.9}$$

A maximum radius r_m has been introduced at which the deflections in the soil are assumed to become vanishingly small [*fig. 4.1*]. Empirically this radius has

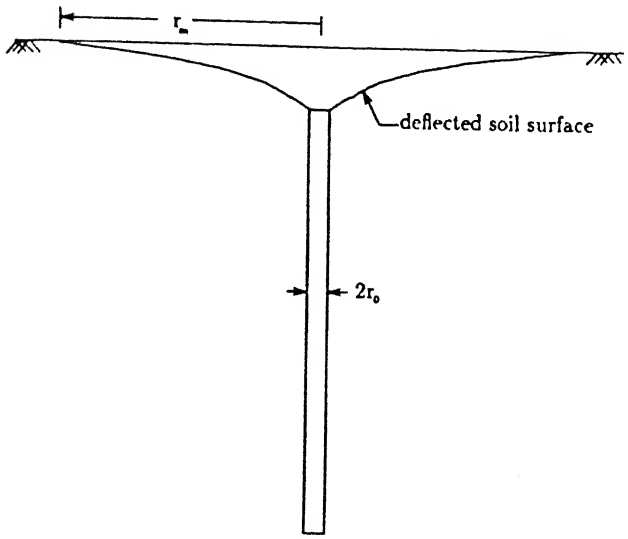


Figure 4.1: Variation of Deflection with Distance from the Pile Axis

been found to be of the order of the length of the pile. The deflection of the pile shaft, w_s is given by

$$w_s = \zeta \frac{\tau_0 r_0}{G} \tag{4.4}$$

where $\zeta = \ln(r_m/r_0)$

The overall load taken by the pile shaft is $P_s = 2\pi r_0 l \overline{\tau_0}$, where $\overline{\tau_0}$ is the average shear stress mobilized at the pile shaft. Thus the load-settlement ratio (or stiffness of the soil-pile system) is

$$\frac{P_s}{w_s} = \frac{2\pi l \overline{G}}{\zeta} \tag{4.5}$$

where $\overline{G}$ is the average shear modulus of the soil over the depth of penetration of the pile.

At the pile base level, the base has been treated as a rigid punch acting at the surface of the soil medium and deflection is obtained as

$$w_b = \frac{P_b}{r_b G_b} \frac{(1 - \nu)}{4} \tag{4.6}$$

where the subscript b refers to the pile base.

For a stiff pile, the base settlement and the shaft settlement will be similar to the settlement of the pile head, w_t. The total load, P_t, may thus be written as

$$P_t = P_b + P_s = w_t\left(\frac{P_b}{w_b} + \frac{P_s}{w_s}\right) \tag{4.7}$$

The response of the pile is then expressed in dimensionless form making use of *equations 4.5* and *4.6* in *equation 4.7* as

$$\frac{P_t}{w_t r_0 G_b} = \frac{4 r_b G_b}{(1 - \nu) r_0 G_b} + \frac{2\pi \overline{G}}{G_b} \frac{l}{r_0} \tag{4.8}$$

Accounting for shaft compression of the pile we get

$$\varepsilon_z = -\frac{dw}{dz} = \frac{P}{\pi r_0^2 E_p} \tag{4.9}$$

where E_p is the modulus of elaticity of the pile.

The load P will vary down the length of the pile as load is shed into the surrounding soil so that

$$\frac{dP}{dz} = -2\pi r_0 \tau_0 \tag{4.10}$$

Finally, w_s and τ_0 may be related by *equation 4.4* to give

$$\frac{d^2 w}{dz^2} = \frac{2G}{\zeta E_p r_0^2} w \tag{4.11}$$

Solving the differential equation and substituting the appropriate boundary conditions at the pile base an expression for the load settlement ratio of the pile head is obtained,

$$\frac{P_t}{G_b r_0 w_t} = \frac{\frac{4\eta}{(1 - \nu)\zeta} + \frac{2\pi\rho}{\zeta} \frac{\tanh(\mu l)}{\mu l} \frac{l}{r_0}}{1 + \frac{4\eta}{\pi\lambda(1 - \nu)\xi} \frac{\tanh(\mu l)}{\mu l} \frac{l}{r_0}} \tag{4.12}$$

where the various dimensionless parameters are,

$\eta = r_b/r_0$ (ratio of underream for underreamed piles)

$\xi = G_l/G_b$ (ratio of end-bearing for end-bearing piles)

$\rho = \overline{G}/G_l$ (variation of soil modulus with depth)

$\lambda = E_p/G_l$ (pile-soil stiffness ratio)

$\zeta = \ln(r_m/r_0)$ (measure of radius of influence of pile)

$\mu l = \sqrt{2/\zeta\lambda}(l/r_0)$ (measure of pile compressibility).

The solution of the pile may be easily adapted to a layered profile by treating each layer seperately and retaining compatibility of the pile displacements between each section. Effectively, the load settlement ratio for the pile base term (the term $4/(1 - \nu)$ in *equation 4.12*) is replaced by the load settlement ratio for the section of the pile below that which is currently being considered as shown in *figure 4.2*.

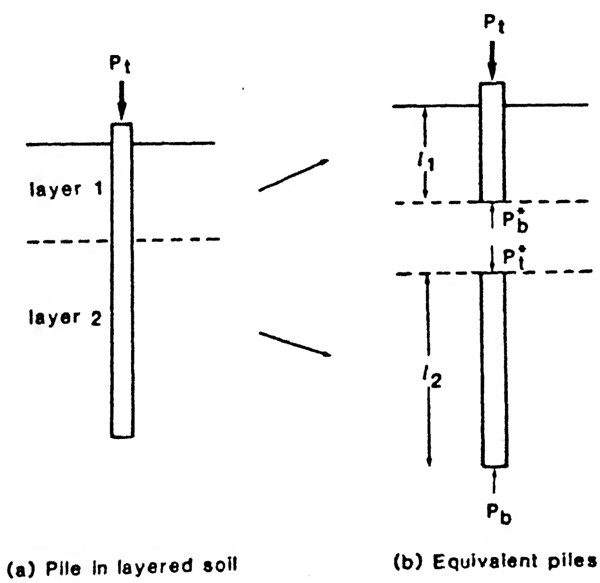


Figure 4.2: Modelling of a Pile in a Layered Soil

4.3 Settlement under Axial Load - Pile Group

The settlement of a pile group is the sum of the immediate or elastic settlement ρ_i and the long-term or consolidation settlement ρ_c. The general equation for the calculating ρ_i for a flexible foundation at the ground surface level is

$$\rho_i = q_n \, 2B \left(\frac{1 - m^2}{E_u} \right) I_p \tag{4.13}$$

where ρ_i is the settlement at the center of the flexible loaded area,

q_n is the net foundation pressure,

B is the width of an equivalent flexible raft,

ν is the Poisson's ratio,

I_p is an influence factor,

and E_u is the deformation modulus for the undrained loading conditions.

4.3.1 Methods of analysis

Using *equation 4.13* the values of I_p depend on the ratio H/B of the depth of the compressible soil layer to the pile group width and on the length to width ratio L/B of the pile group.

However, it is found to be more convenient to use the expression given by Janbu, Bjerrum and Kjaernsli [Tomlinson (1978)] to obtain the average immediate settlement of a foundation at depth D below the surface where

equivalent raft foundation, are shown in *figure 4.3.*

The base of the equivalent raft for different cases of soil conditions is as shown in *figure 4.4*

In layered soils with different values of the deformation modulus E_u in each layer, the strata below the base of the equivalent raft are divided into a number of representative horizontal layers and an average value of E_u is assigned to each layer. The dimensions L and B in *figure 4.3* are determined on the assumption that the load is spread to the surface of each layer at anangle of 30^o from the edges of the equivalent raft . The total settlement of the pile foundation is then the sum of the average settlements calculated for each soil layer from *equation 4.14*.

The computation of the settlement of a group of piles if often done by the application of influence factors. There are many such influence factors in literature. Skempton (1953) has suggested a relationship between the settlement of a pile group and a single pile. Pitchumani and D'Appolonia (1967), Poulos (1968) have suggested methods of obtaining such influence factors on the basis of the theory of elasticity. Randolph and Wroth (1979) have suggested influence factors as an extension of the theory for single pile given in *section 4.1.3.*

4.3.2 Method Adopted

Although it might be thought more prudent to maintain a consistency in the method of analysis used for a single pile and a pile group, the method suggested by Randolph and Wroth (1979) suffers from the drawback that seperate influence

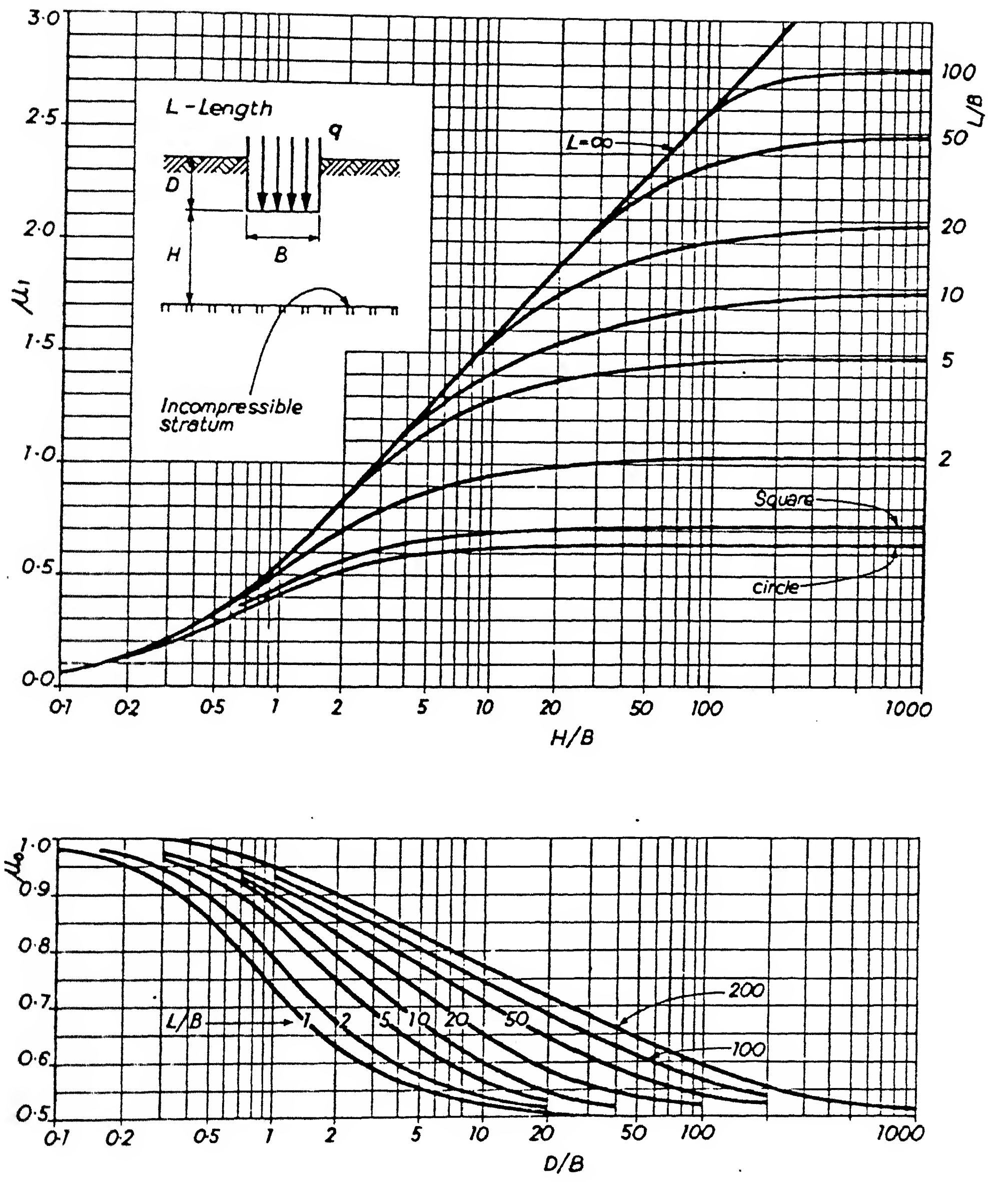
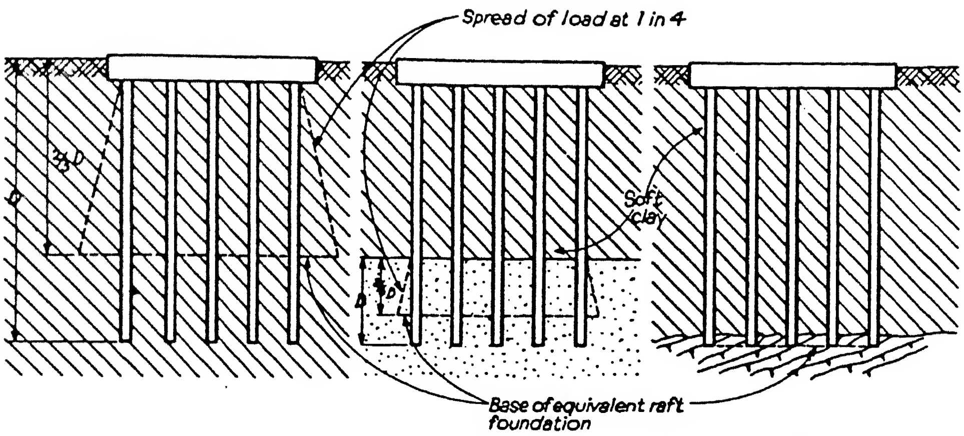


Figure 4.3: Influence Factors for Calculating the Elastic Settlements of Pile Groups



(a) Group of piles supported predominantly by skin friction
(c) Group of piles supported in end bearing on hard rock stratum
(b) Group of piles driven through soft clay to combined skin friction and end bearing in stratum of dense granular soil

Figure 4.4: Base of Equivalent Raft for Different Cases of Soil Conditions

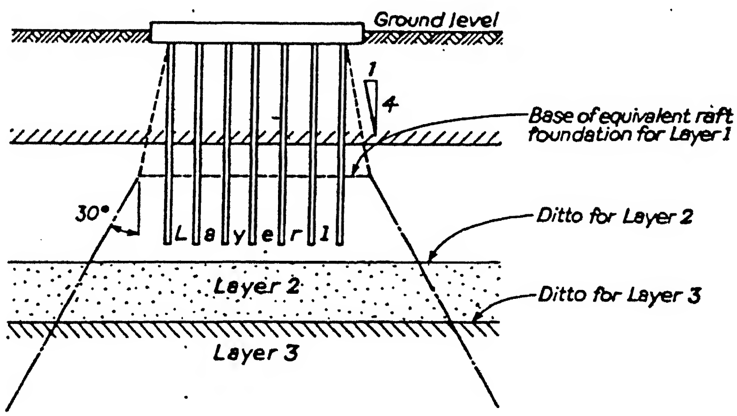


Figure 4.5: Load Distribution Beneath Pile Group in Layered Soil Formation

factors have been given for the shaft and base capacities. Although this might be considered to be a better representation of the problem, it can be easily appreciated that the determination of the contribution of the shaft and base capacity to the total capacity is not a simple task. Hence the influence factors suggested by Poulos (1968) have been used for the analysis.

The influence factors have been obtained by Poulos (1968) by the integration of the Mindlin's equation for the vertical displacement in a semi-infinite mass resulting

from an interior vertical loading. These influence factors have been plotted by Poulos and Mattes and these are reproduced by Poulos (1980), *figure 4.6* for $L/d = 10$, *figure 4.7* for $L/d = 25$, *figure 4.8* for $L/d = 50$ and *figure 4.9* for $L/d = 100$, where L/d is the length of pile/diameter of pile, for different values of s/d (spacing of the piles/diameter of pile) ratios and different values of soil stiffness K at a value of poisson's ratio of the soil, $\nu_s = 0.5$.

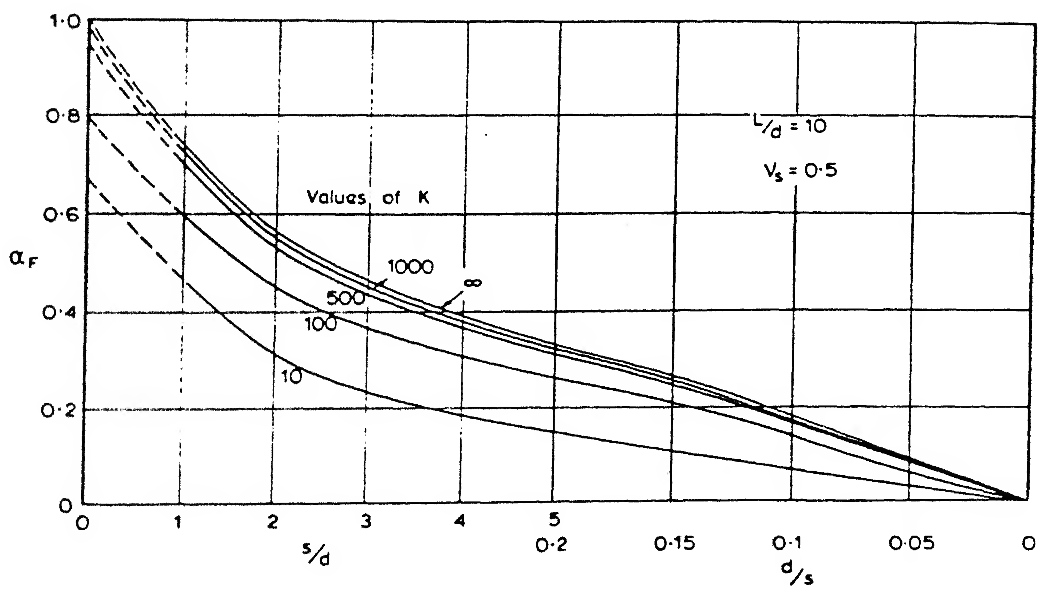


Figure 4.6: Interaction Factors for Floating Piles, L/d = 10

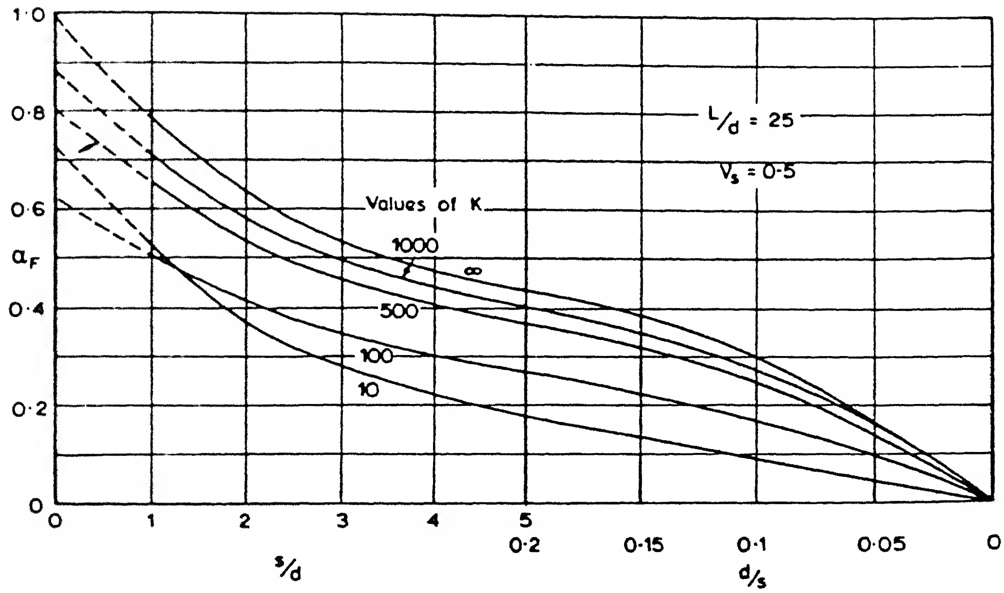


Figure 4.7: Interaction Factors for Floating Piles, L/d = 25

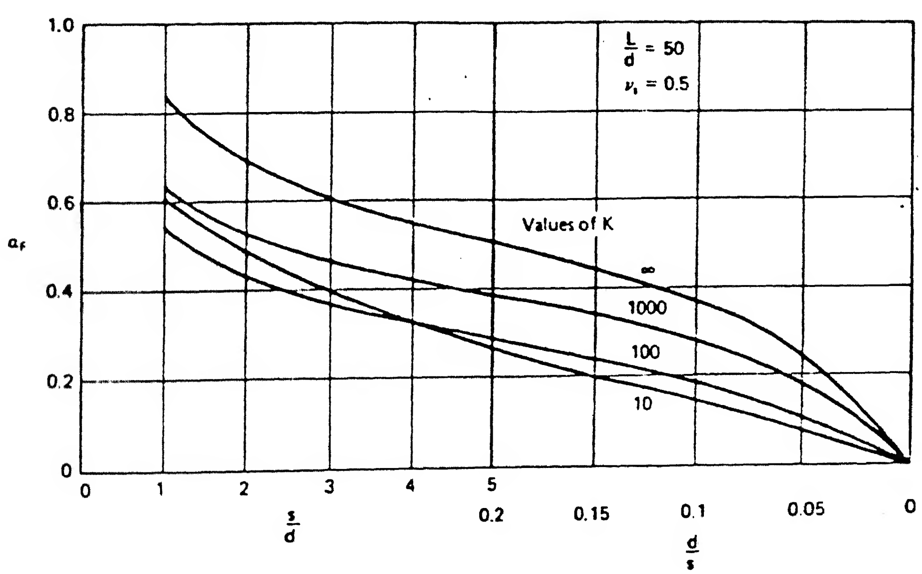


Figure 4.8: Interaction Factors for Floating Piles, L/d = 50

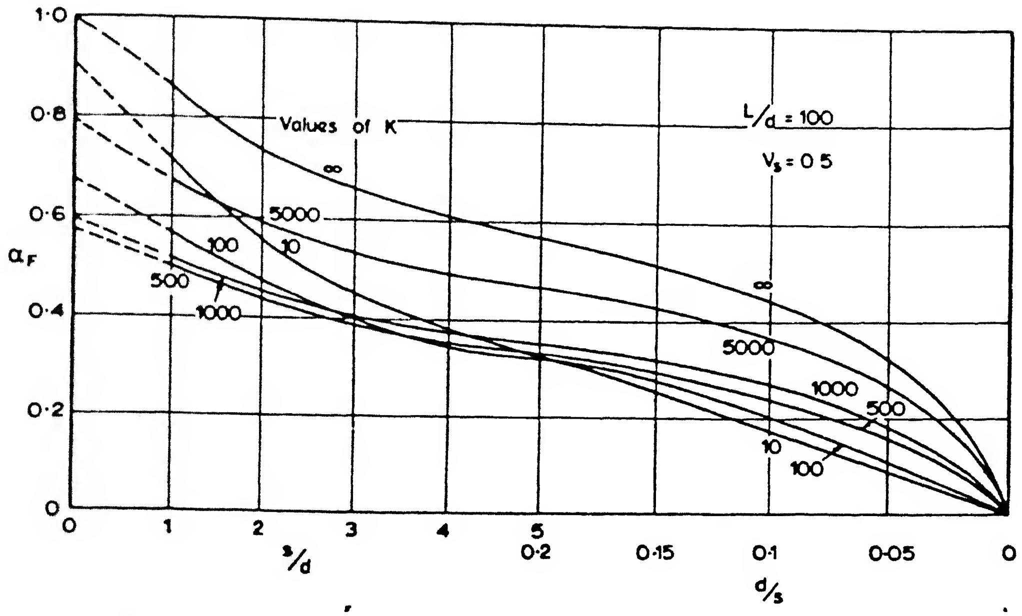


Figure 4.9: Interaction Factors for Floating Piles, L/d = 100

Since these values are obtained from an equation a load acting in a semi-infinite media, curves for correction factor, N_h for the effect of finite layer have been given (*figure 4.10*).

Also since these curves are plotted at a poisson's ratio $\nu_s = 0.5$, curves for correction factor, N_ν for the effect of poisson's ratio have been given *figure 4.11*.

4.4 Settlement under Lateral Load

Since the computation of the lateral capacity of the pile by the beam on elastic foundation approach is made on the basis of the permissible lateral deflections *section 3.1.3* the lateral settlements need not be computed since the lateral deflections of the pile will be within the permissible. No differentiation is made between the concept of group lateral deflection and the lateral deflection of a single pile.

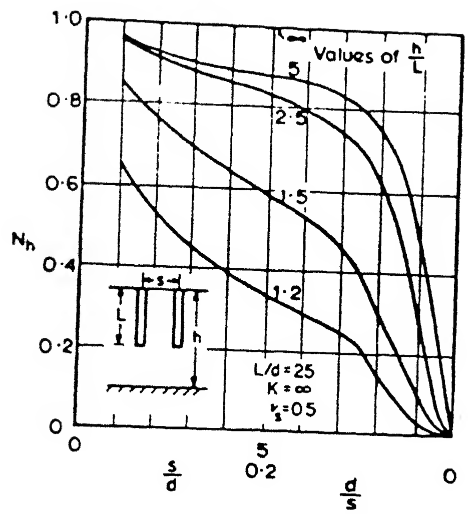


Figure 4.10: Correction Factor N_h to Interaction Factors for the Effect of Finite Layer Depth

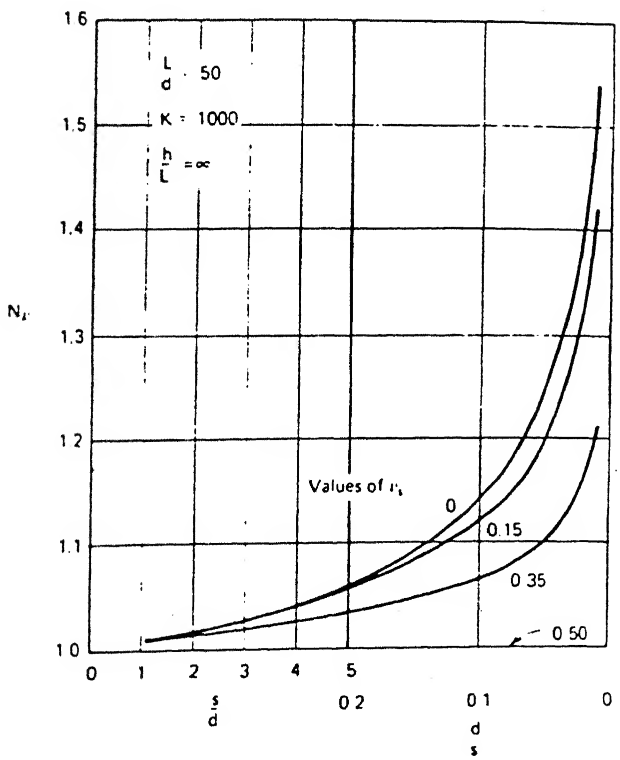


Figure 4.11: Correction Factor N_ν to Interaction Factors for the Effect of Poisson's Ratio

Chapter 5

Loads and Pile Configuration

The developed expert system (PILEAN) works for any number of loads coming on the structure and selects a configuration of the pile group in accordance with the loads that the pile group has to cater for. This is achieved by a set of heuristics inherent in the program. This chapter deals with the principles on which the heuristics is based. The first section gives the handling of the loads to make it more amenable for the choice of a pile group, the second section deals with how an initial selection of a pile group is made and the final section deals with the checks imposed on the group and how a final selection is made.

5.1 Loads

The loads are specified in the form of a point of action of the load, the x, y and z components of the forces and the x, y and z components of the moments. The axis that is chosen by the user to specify the loads is not important since the loads are transformed so that the origin passes through the center of the load system.

5.1.1 Transformation of the axes of the load system

There are distinct advantages of transforming the axis so that the origin passes through the center of the load system. These advantages are listed below,

1. The major advantage of such a transformation is the alignment of the pile group. Once the loads have been so transformed, the pile group can be placed so that the length and breadth of the pile group are in conformity with the spatial distribution of the loads. An alignment in variation to this would result in a pile group spread over an extent that is much more than is required. This can be easily appreciated by the arrangemnets of piles in *figure 5.1* case(a) and case(b).

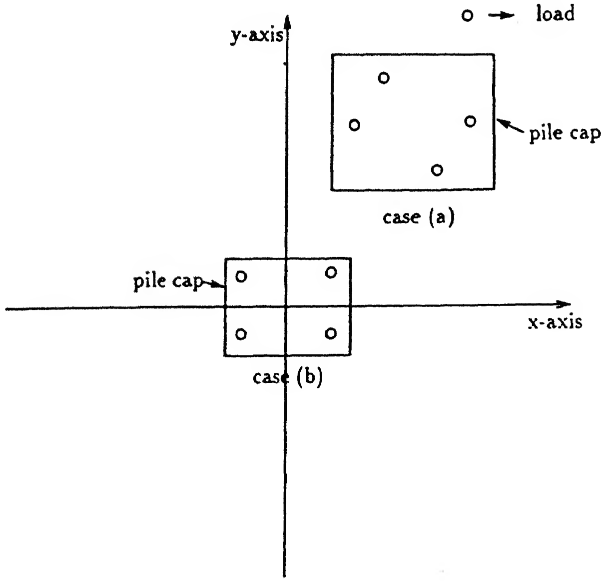


Figure 5.1: Extent of Pile Group Required (seen from size of pile cap) for Loads without (case(a)) and with (case(b)) Transformation of Axes

2. It is also important that the center of the pile group coincide with the center of the load system to minimize the difference in loads coming on the

individual piles in the group. This is very important in order to optimize the piles used.

3. Such a transformation also helps in the definition of the pile in the program. The pile group can all be defined with the center of pile group at the origin.

The process of transformation is achieved by first rotating the loads about the origin so that the sum of the squares of the distances of the loads from the axes is a minimum. The loads are then laterally shifted so that the center of the load system coincides with the origin. In such a case the loads and moments have also to be suitably transformed. Sometimes due to excessive moments or excessive differences in the loads the center of the force system may be very far away from the geometric center of the points of application of the loads. In such a case the center has to be limited to the middle third of the of the area over which the loads are spread or else the moments coming on the pile cap may become excessive.

5.2 Pile Configuration

The number of piles can be easily selected on the basis of the axial and lateral loads on the system and the axial and lateral capacities of the piles.

There are two criteria which govern the initial selection of the pile configuration,

1. The pile group should be oriented such that the length of the group lies in the direction perpendicular to the axis having a greater moment. Since the piles have now been oriented with respect to the loads, this leaves us

to choose only between two perpendicular directions.

2. The pile group should be oriented such that the length of the group lies as far as possible along the length of the load system.

On the basis of these two criteria an initial number is assigned according to which the program chooses an appropriate length to width ratio of the group. Presently the program has a database for the coordinates of upto 22 piles. This number can very easily be raised to any desired figure.

5.3 Checks Imposed on the pile group

Checks are imposed on the selected pile group in stages. Initially a check for the extent of the load and the pile group is imposed in order to ensure that the loads do not come on the pile cap at a point outside the pile group. This would result in excessive moments in the pile cap.

Next, checks are imposed such that the individual piles have adequate factor of safety against axial load, lateral load, moments and combined axial and lateral load. The axial load coming on a pile is calculated by the expression

$$P_{vi} = \frac{P_z}{n} \pm \frac{M_y \, y_i \, A_i}{I_x} \pm \frac{M_x \, x_i \, A_i}{I_y} \tag{5.1}$$

where P_{vi} is the axial load coming on the i^{th} column

P_z is the resultant of all the vertical loads

M_x is the moment about the x-axis

M_y is the moment about the y-axis

x_i, y_i are the X and Y coordinates of the point of application of the load

I_x, I_y are the moment of inertia about the x and y axes respectively.

where I_x and I_y are obtained as given below,

$$I_x = A_1 y_1^2 + A_2 y_2^2 + \cdots + A_n y_n^2$$

$$I_y = A_1 x_1^2 + A_2 x_2^2 + \cdots + A_n y_n^2$$

$$I_z = I_x + I_y$$

$$P_{hxi} = \frac{P_x}{n} \pm \frac{M_z \, y_i \, A_i}{I_z} \tag{5.2}$$

$$P_{hyi} = \frac{P_y}{n} \pm \frac{M_z \, x_i \, A_i}{I_z} \tag{5.3}$$

where P_{hxi} is the lateral load coming on the i^{th} column in the x-dirn.

P_{hyi} is the lateral load coming on the i^{th} column in the y-dirn.

P_x is the resultant of all the horizontal x-directional loads

P_y is the resultant of all the horizontal y-directional loads

M_z is the moment about the z-axis

x_i, y_i are the x and y coordinates of the point of application of the load

A_i is the area of the i^{th} pile.

The safety against combined axial and lateral load can be checked by the equation for a column under combined axial and lateral load [IS:456-1978].

$$\frac{\sigma'_{cc}}{\sigma_{cc}} + \frac{\sigma'_{cbc}}{\sigma_{cbc}} \leq 1.0 \tag{5.4}$$

After this the pile group is checked for group capacities. Finally a check on the settlements of the individual piles is made.

It is possible, in fact highly probable, that after all these checks there are still more than than one pile diameter in 'contention'. In order to make a final choice a representative economic analysis is made. Three factors are considered in the economic analysis, *(a)* the quantity of concrete, *(b)* the quantity of steel and *(c)* additional costs per unit length of the pile which could be interpreted as the cost of installation of the pile. The program also gives the result of the economic analysis as the *cost of piling*. Although these figures may be representative of the comparative costs of the installation of the various diameters of pile considered, it may not be equal to the actual cost incurred in the piling.

6.2.1 Input File – col.in

no. of loads (n)

x coord of load 1, y coord of load 1

force in x-dirn, force in y-dirn, force in z-dirn

moment in x-dirn, moment in y-dirn, moment in z-dirn

x coord of load 2, y coord of load 2

⋮

x coord of load n, y coord of load n

force in x-dirn, force in y-dirn, force in z-dirn

moment in x-dirn, moment in y-dirn, moment in z-dirn

- All the distances are to be declared in metres, forces in kN and moments in kN-m.
- Any system of axes can be selected for the definition of the loads.
- A declaration of number of loads in line 1 of *col.in* greater than the number listed after the declaration is likely to cause an abrupt end to the run.

6.2.2 Input File – pile.in

pile depth

shape of pile

driven/bored, precast/cast in-situ

depth of bedrock

depth of water table

grade of concrete, yield strength of reinforcement steel

1

- Any number can be entered for the entry *pile depth.*

- Shape of the pile can be specified as Circular/ Rectangular/ Octagonal/ Hexagonal.

- If the pile is driven - enter 1, if bored - enter 0.

- If the pile is precast - enter 1, if cast in-situ - enter 0.

- Depth of bedrock and depth of water table are to be specified in metres.

- Grade of concrete can be specified as 20, 25 etc. for M20, M25, ... grades.

- The yield stress of the reinforcement steel has to be specified in N/mm^2 as 250, 415 or 500.

- The final entry of 1 represents the choice of method used as the use of bearing capacity equations. Since there is no other alternative presently incorporated in the system, this number is invariant.

6.2.3 Input File – soil.in

no. of layers n_l

depth of layer 1

c , c'

ϕ, ϕ'

γ, γ_{sub}

(if ϕ > 10.0)

OCR

consistency index

(if ϕ < 10.0)

relative density

ϕ_{cv}

depth of layer 2

⋮

depth of layer n_l

⋮

- Here again the number of layers whose properties are entered should be at least equal to n_l.
- c, c' are the drained and undrained values of cohesion of the soil, in kN/m^2.
- ϕ, ϕ' are the drained and undrained values of the friction angle, in degrees.
- γ, γ_{sub} are the unit weight and submerged unit weight of the soil respectively, in kN/m^3.
- OCR is the overconsolidation ratio of the cohesive soil.
- ϕ_{cv} is the friction angle at constant volume (when soil shears with zero dilation), in degrees.

6.2.4 Input File – lat.in

Do you have a value of the horizontal modulus of subgrade reaction for soil layer 1(y/n) ?

If yes, enter value

⋮

Do you have a value of the horizontal modulus of subgrade reaction for soil layer n_l (y/n) ?

If yes, enter value

Do you have a value for the permissible lateral deflection (y/n) ?

If yes, enter value

- The value of the horizontal modulus of subgrade reaction is to be declared in kN/m^2.

- One can answer in the negative to the question about the horizontal modulus of subgrade reaction in all cases except for a soft soil. For all other soils if the answer is negative a suitable value of the horizontal modulus of subgrade reaction is assumed (*table 6.1* for clays and *table 6.2* for cohesionless soils).

- The permissible lateral deflection is to be declared in metres. If not declared it is assumed as *dia_pile*/75.0.

In the following pages are given the input and output files for three runs of the expert system.

Consistency	Stiff	Very stiff	Hard
Undrained cohesion (c_u)			
kN/m^2	100–200	200–400	>400
$tons/ft^2$	1–2	2–4	>4
Range of k_1			
MN/m^3	18–36	36–72	>72
$tons/ft^3$	50–100	100–200	>200
Recommended k_1			
MN/m^3	27	54	>108
$tons/ft^3$	75	150	>300

Table 6.1: Relationship of Horizontal Modulus of Subgrade Reaction to Undrained Shearing Strength of Stiff Overconsolidated Clay

Relative density	Loose	Medium dense	Dense
n_h for dry or moist soil (Terzaghi)			
MN/m^3	2.5	7.5	20
$tons/ft^3$	7	21	56
n_h for submerged soil (Terzaghi)			
MN/m^3	1.4	5	12
$tons/ft^3$	4	14	34
n_h for submerged soil (Reese *et al.*)			
MN/m^3	5.3	16.3	34
$tons/ft^3$	15	46	96

Table 6.2: Factors for Calculating Coefficient of Modulus Variation (n_h) for Cohesionless Soils

RUN 1

```
DO YOU WISH TO CHANGE THE LOADS (Y/N) y

Consider a right-handed co-ordinate system with the z-axis vertically upwards
No of loads presently acting on the structure = 4
Enter new value for the number of loads
4

LOAD 1
          Co-ordinates of Load 1 are 4.00 m; 6.00 m
          Enter new co-ordinates (m)
          4 0 6 0
          Load in the x,y,z directions are 15.00 kN; 0.00kN; -2000.00 kN
          Enter new loads (kN)
          15 0 0 -2000 0
          Moments in the x,y,z directions are 0.00 kN-m; 0.00 kN-m; 0.00 kN-m
          Enter new moments (kN-m)
          0 0 0 0 0.0

LOAD 2
          Co-ordinates of Load 2 are 6.00 m; 6.00 m
          Enter new co-ordinates (m)
          6 0 6 0
          Load in the x,y,z directions are 25.00 kN; 0.00kN; -1500.00 kN
          Enter new loads (kN)
          25 0 0 0 -1500.0
          Moments in the x,y,z directions are 0.00 kN-m; 0.00 kN-m; 0.00 kN-m
          Enter new moments (kN-m)
          0 0 0.0 0.0

LOAD 3
          Co-ordinates of Load 3 are 6.00 m; 4.00 m
          Enter new co-ordinates (m)
          6.0 4.0
          Load in the x,y,z directions are 10.00 kN; 0.00kN; -1500.00 kN
          Enter new loads (kN)
          10.0 0.0 -1500.0
          Moments in the x,y,z directions are 0.00 kN-m; 0.00 kN-m; 0.00 kN-m
          Enter new moments (kN-m)
          0.0 0.0 0.0

LOAD 4
          Co-ordinates of Load 4 are 4.00 m; 4.00 m
          Enter new co-ordinates (m)
          4 0 4 0
          Load in the x,y,z directions are 0.00 kN; 0.00kN; -500.00 kN
          Enter new loads (kN)
          0 0 0.0 -500.0
          Moments in the x,y,z directions are 0.00 kN-m; 0.00 kN-m; 0.00 kN-m
          Enter new moments (kN-m)
          0 0 0.0 0.0


DO YOU WISH TO CHANGE THE VALUES OF ANY SITE CONDITIONS (Y/N) y

Present depth of bedrock = 12.00 m
Enter new depth of bedrock (m)
12.0
Present depth of water table = 7.00 m
Enter new depth of water table (m)
7.0
```

```
USE OF BEARING CAPACITY EQUATIONS

DO YOU WISH TO CHANGE THE SOIL PARAMETERS (Y/N) y

Present value of no  of layers = 2
Enter new value of the no  of layers
2
Present value of depth of layer 1 is 8.00 m
Enter new value of layer depth (m)
8.0
Present values of c and cprime are 100.00 kN/m^2 and 105.00 kN/m^2 respectively
Enter new values of c and cprime (kN/m^2)
100.0 105.0
Present values of PHI and PHIprime are 0.00 deg and 0.00 deg respectively
Enter new values of PHI and PHIprime (deg)
0 0
Present values of gamma and gamma_sub are 20.00 kN/m^3and 9.50 kN/m^3 respectively
Enter new values of gamma and gamma_sub (kN/m^3)
20.0 9.5
Do you have the value of the horizontal modulus of subgrade
 reaction of the soil ? (Y/N) Y
Enter the value in kN/m^2
18000
Present value of OCR is 1.10
Enter the value of OCR
1.1
Consistency index is 0.65
Value of consistency index
0.65
Present value of depth of layer 2 is 7.00 m
Enter new value of layer depth (m)
7.0
Present values of c and cprime are 0.00 kN/m^2 and 0.00 kN/m^2 respectively
Enter new values of c and cprime (kN/m^2)
0.0 0.0
Present values of PHI and PHIprime are 25.00 deg and 28.00 deg respectively
Enter new values of PHI and PHIprime (deg)
25.0 28.0
Present values of gamma and gamma_sub are 20.00 kN/m^3and 9.50 kN/m^3 respectively
Enter new values of gamma and gamma_sub (kN/m^3)
20.0 9.5
Do you have the value of the horizontal modulus of subgrade
 reaction of the soil ? (Y/N) N
Present value of relative density is 0.75
Enter the value of relative density
0.75
Present value of PHI at constant volume is 30.00 (deg)
Enter the value of PHI at constant volume (deg)
30.0
THE PERMISSIBLE LATERAL DEFLECTION BY DEFAULT IS SET AT Dia_Pile/75.0
DO YOU HAVE A DIFFERENT CHOICE ? (Y/N) y
ENTER PERMISSIBLE VALUE OF LATERAL DEFLECTION (m)
0.005


DEFAULT GRADE OF CONCRETE TO BE USED IS M30
DO YOU HAVE A DIFFERENT CHOICE ? (Y/N) y
CHOICES AVAILABLE FOR CONCRETE GRADE :
M15
M20
M25
M30
M35
```

```
M45
ENTER YOUR CHOICE - M25

DEFAULT YIELD STRESS OF STEEL TO BE USED IS 250.00
DO YOU HAVE A DIFFERENT CHOICE ? (Y/N) y
CHOICES AVAILABLE FOR STEEL GRADE :
Mild steel(250)
Fe 415 steel)
Fe 500 steel)
ENTER YIELD STRESS -415



WOULD YOU LIKE TO ALTER PREVIOUS PILE SECTIONS ? (Y/N) y
Is the pile driven or bored (D/B) d
Is the pile precast or cast-in-situ (P/C) p
DO YOU HAVE
1  A SPECIFIC PILE SECTION(S) TO USE ?
2  A SPECIFIC PILE DIAMETER THAT YOU WOULD LIKE TO USE
3  NO CHOICE
ENTER CHOICE (1/2/3) = 3
Using ten standard driven precast sections
```

REPORT FOR THE GIVEN SOIL CONDITIONS

SOIL CHARACTERISTICS

LAYER 1

Layer Depth	: 8.00 m		
Cohesion (c)	:100.00 kN/m^2	Undrained cohesion (c')	:105.00 kN/m^2
Friction (phi)	: 0.00 deg	Undrained friction (phi')	: 0.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
O C R	: 1.10		
Consistency Index	: 0.65		

LAYER 2

Layer Depth	: 7.00 m		
Cohesion (c)	: 0.00 kN/m^2	Undrained cohesion (c')	: 0.00 kN/m^2
Friction (phi)	: 25.00 deg	Undrained friction (phi')	: 28.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
Relative density	: 0.75		
Phi at const. vol.	: 30.00 deg		

LOADS MODIFIED TO HAVE THE ORIGIN AS CENTER

	X	Y	Z
Forces	15.783	-47.444	-5500.000
Moments	0.001	-0.002	-16.364

Depth of bedrock : 12.00 m

Depth of water table : 7.00 m

Grade of concrete : M25 Yield strength of steel reinforcement : 415.0 N/mm^2

PILE CHARACTERISTICS

Pile is driven precast

Length of the pile : 12.00 m

Diameter or C/S of pile	Axial load carrying capacity	Lateral load carrying capacity
Rectangular: 0.30 m	363.57 kN	6.61 kN
Rectangular: 0.50 m	935.24 kN	59.15 kN
Circular: 0.30 m	272.59 kN	9.49 kN
Circular: 0.50 m	703.36 kN	48.44 kN
Rectangular: 0.60 m	1342.50 kN	97.13 kN
Circular: 0.60 m	1003.80 kN	64.95 kN
Rectangular: 0.80 m	2476.37 kN	42.84 kN
Rectangular: 1.00 m	3661.87 kN	49.02 kN
Circular: 0.80 m	1845.11 kN	126.63 kN
Circular: 1.00 m	2876.02 kN	54.67 kN

STATUS OF CONSIDERED PILES

	Status	Eco. crit. 1	Eco. crit. 2	Eco. crit. 3	Cost(Rs)
Pile 1	Accepted	1.44	2.28	16.00	140774.06
Pile 2	Accepted	2.00	2.41	8.00	104216.08

Pile 3 Rejected				
Pile 4 Accepted	1.57	2.21	8.00	97015.88
Pile 5 Accepted	1.80	2.35	5.00	83550.00
Pile 6 Accepted	2.26	2.77	8.00	112322.75
Pile 7 Accepted	1.92	1.41	3.00	55963.86
Pile 8 Rejected				
Pile 9 Accepted	1.51	2.12	3.00	64870.09
Pile 10 Rejected				

CHOSEN PILE
Rectangular pile of side 0.80 m. with 4 nos. of long. reinforcing bars of 40.00 dia.
No. of piles = 3
Cost of piling = Rs.55963.86

x(m)	y(m)	Vert.load(kN)	X-load (kN)	Y-load (kN)	axial defl.(m)
0.000	1.960	1833.33	12.06	-15.81	0.0063
-1.350	-0.980	1833.33	1.86	-11.12	0.0064
1.350	-0.980	1833.33	1.86	-20.50	0.0064

Axial capacity of the pile group = 7428.80 kN
Lateral capacity of the pile group = 128.51 kN

RUN 2

col.in

```
4
4.000000 6.000000
105.000000 0.000000 -200.000000
0.000000 0.000000 0.000000
6.000000 6.000000
250.000000 0.000000 -15.000000
0.000000 0.000000 0.000000
6.000000 4.000000
100.000000 0.000000 -15.000000
0.000000 0.000000 0.000000
4.000000 4.000000
40.000000 0.000000 -50.000000
0.000000 0.000000 0.000000
```

pile.in

```
12.000000
    Circular
1 0
12.000000
7.000000
25.000000 250.000000
1
1.000000
10
```

soil.in

```
2
8.000
120.000 125.000
0.000 0.000
20.000 9.500
1.050
0.650
7.000
130.000 132.000
5.000 8.000
20.000 9.500
1.100
0.700
```

lat.in

```
n
N
Y
0.005000
```

REPORT FOR THE GIVEN SOIL CONDITIONS

SOIL CHARACTERISTICS

LAYER 1

Layer Depth	: 8.00 m		
Cohesion (c)	:120.00 kN/m^2	Undrained cohesion (c')	:125.00 kN/m^2
Friction (phi)	: 0.00 deg	Undrained friction (phi')	: 0.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
O C R	: 1.05		
Consistency Index	: 0.65		

LAYER 2

Layer Depth	: 7.00 m		
Cohesion (c)	:130.00 kN/m^2	Undrained cohesion (c')	:132.00 kN/m^2
Friction (phi)	: 5.00 deg	Undrained friction (phi')	: 8.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
O C R	: 1.10		
Consistency Index	: 0.70		

LOADS MODIFIED TO HAVE THE ORIGIN AS CENTER

	X	Y	Z
Forces	-408.917	-278.948	-280.000
Moments	0.000	6.874	43.331

Depth of bedrock : 12.00 m

Depth of water table : 7.00 m

Grade of concrete : M25 Yield strength of steel reinforcement : 250.0 N/mm^2

PILE CHARACTERISTICS

Pile is driven cast-in-situ
Length of the pile : 12.00 m

Diameter or C/S of pile	Axial load carrying capacity	Lateral load carrying capacity
Rectangular: 0.30 m	242.63 kN	8.19 kN
Rectangular: 0.50 m	424.73 kN	73.68 kN
Circular: 0.30 m	200.97 kN	10.59 kN
Circular: 0.50 m	333.58 kN	53.90 kN
Rectangular: 0.60 m	537.76 kN	116.12 kN
Circular: 0.60 m	422.36 kN	71.83 kN
Rectangular: 0.80 m	791.89 kN	133.99 kN
Rectangular: 1.00 m	1083.47 kN	66.19 kN
Circular: 0.80 m	621.95 kN	136.88 kN
Circular: 1.00 m	850.95 kN	65.69 kN

STATUS OF CONSIDERED PILES

	Status	Eco. crit. 1	Eco. crit. 2	Eco. crit. 3	Cost(Rs)
Pile 1	Rejected				
Pile 2	Accepted	2.00	2.41	8.00	103734.30

Pile 3 Rejected				
Pile 4 Accepted	1.57	2.21	8.00	96574.79
Pile 5 Accepted	2.88	3.76	8.00	132927.22
Pile 6 Accepted	2.26	2.77	8.00	111769.47
Pile 7 Accepted	2.56	1.88	4.00	74242.09
Pile 8 Accepted	7.00	3.29	7.00	145103.98
Pile 9 Accepted	1.51	2.12	3.00	64446.65
Pile 10 Accepted	5.50	4.94	7.00	163117.30

CHOSEN PILE
Circular pile of diameter 0.80 m.
with 6 nos. of long. reinforcing bars of 40.00 dia.
No. of piles = 3
Cost of piling = Rs.64446.65

x(m)	y(m)	Vert.load(kN)	X-load (kN)	Y-load (kN)	axial defl.(m)
0.000	1.740	93.33	-152.98	-92.98	0.0008
-1.500	-0.870	91.04	-127.97	-107.36	0.0008
1.500	-0.870	95.62	-127.97	-78.61	0.0008

Axial capacity of the pile group = 1865.60 kN
Lateral capacity of the pile group = 410.65 kN

RUN 3

col.in

```
4
4.000000 6.000000
5.000000 0.000000 -2000.000000
10.000000 0.000000 0.000000
6.000000 6.000000
0.000000 0.000000 -750.000000
0.000000 20.000000 0.000000
6.000000 4.000000
0.000000 0.000000 -1050.000000
30.000000 0.000000 0.000000
4.000000 4.000000
0.000000 0.000000 -500.000000
20.000000 0.000000 0.000000
```

pile.in

```
12.000000
    Circular
0 1
12.000000
7.000000
25.000000 250.000000
1
0.750000
8
```

soil.in

```
3
8.000
120.000 125.000
0.000 0.000
20.000 9.500
1.050
0.650
7.000
100.000 102.000
15.000 18.000
20.000 9.500
0.550
18.000
5.000
0.000 0.000
25.000 28.000
20.000 9.500
0.650
30.000
```

lat.in

```
 n
 N
 n
 Y
0.005000
```

REPORT FOR THE GIVEN SOIL CONDITIONS

SOIL CHARACTERISTICS

LAYER 1

Layer Depth	: 2.00 m		
Cohesion (c)	:120.00 kN/m^2	Undrained cohesion (c')	:125.00 kN/m^2
Friction (phi)	: 0.00 deg	Undrained friction (phi')	: 0.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
O C R	: 1.05		
Consistency Index	: 0.65		

LAYER 2

Layer Depth	: 7.00 m		
Cohesion (c)	:100.00 kN/m^2	Undrained cohesion (c')	:102.00 kN/m^2
Friction (phi)	: 15.00 deg	Undrained friction (phi')	: 18.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
Relative density	: 0.55		
Phi at const. vol.	: 18.00 deg		

LAYER 3

Layer Depth	: 5.00 m		
Cohesion (c)	: 0.00 kN/m^2	Undrained cohesion (c')	: 0.00 kN/m^2
Friction (phi)	: 25.00 deg	Undrained friction (phi')	: 28.00 deg
Unit weight (gamma)	: 20.00 kN/m^3	Sub. unit weight (gamma_sub)	: 9.50 kN/m^3
Relative density	: 0.65		
Phi at const. vol.	: 30.00 deg		

LOADS MODIFIED TO HAVE THE ORIGIN AS CENTER

	X	Y	Z
Forces	-2.545	-4.304	-4300.000
Moments	-3.889	72.006	-3.744

Depth of bedrock : 12.00 m

Depth of water table : 7.00 m

Grade of concrete : M25 Yield strength of steel reinforcement : 250.0 N/mm^2

PILE CHARACTERISTICS

Pile is bored precast
Length of the pile : 12.00 m

Diameter or C/S of pile	Axial load carrying capacity	Lateral load carrying capacity
Circular: 0.75 m	588.84 kN	23.70 kN

STATUS OF CONSIDERED PILES

	Status	Eco. crit. 1	Eco. crit. 2	Eco. crit. 3	Cost(Rs)
Pile 1	Accepted	3.53	1.41	8.00	99417.02

CHOSEN PILE
Circular pile of diameter 0.75 m.

with 6 nos. of long. reinforcing bars of 20.00 dia.
No. of piles = 8
Cost of piling = Rs 99417.02

x(m)	y(m)	Vert.load(kN)	X-load (kN)	Y-load (kN)	axial defl.(m)
-2.250	0.000	530.39	-0.32	-0.17	0.0063
0.000	0.000	537.50	-0.32	-0.54	0.0069
2.250	1.960	544.94	0.00	-0.91	0.0057
-1.120	1.960	534.28	0.00	-0.35	0.0066
1.120	1.960	541.39	0.00	-0.72	0.0060
-2.250	-1.960	530.06	-0.64	-0.17	0.0060
0.000	-1.960	537.17	-0.64	-0.54	0.0064
2.250	-1.960	544.28	-0.64	-0.91	0.0063

Axial capacity of the pile group = 4284.00 kN
Lateral capacity of the pile group = 189.64 kN

Chapter 7

Conclusions

From the results of the expert system shown in *chapter 6*, it can be seen that the expert system functions well for a vareity of loading, soil conditions and pile choices. It is hoped that the expert system (PILEAN) developed in this thesis will prove an aid to designers in the future who wish to design pile groups and that it will simplify the task for them. The rest of this chapter will be devoted to the enumeration of the limitations of the expert system presented in this thesis and the scope for further improvements both in this expert system as well as the scope for the development of similar expert systems in other fields.

7.1 Limitations

A few of the limitations of the expert system (PILEAN) developed in this thesis are listed,

1. With the aid of the present database only upto a group of 22 piles can be designed.

2. The system considers a layered soil profile and assumes that the same soil profile exists over the entire extent of the pile group.

3. The system considers all piles in a group to be of the same specification.

4. The system does not consider the effect of negative skin friction on the capacity of the piles.

5. It does not compute the axial load carrying capacity of piles subjected to uplift forces and hence cannot be used for the design of piles/pile groups subjected to uplift forces.

6. It calculates only the longitudinal reinforcement required for a pile.

7. It considers the effect of the combined axial and lateral load only in terems of the structural capacity of the pile section.

8. It does not have a check at every stage for wrong entries and may end up giving erroneous results for wrong entries in certain input values.

Although the expert system works well for a vareity of cases as demonstrated in *chapter 6*, there may be some case which may not be accounted for. Such errors come to light only with the use of the system over a period of time.

7.2 Scope for further work

Expert systems generally mature over the years as capabilities are added and bugs are eliminated. Hence there is always a scope for further work on any expert system fro quite some time after it has been developed. Likewise, a few improvements that could be made to the expert system (PILEAN) developed in this thesis are listed,

1. The present expert system works only for concrete piles. It could be extended to other types of piles (*section 1.2*).

2. The database of pile groups could be extended beyond the 22 pile group that ot presently contains.

3. The capacity of the pile against uplift forces could be calculated.

4. The present expert system takes the laboratory test results as input and utilizes the static bearing capacity equations to compute the axial capacity of the pile. The capacity of a pile could also be calculated by other methods as mentioned in *section 2.1* such as the use of Standard Penetration Test (SPT) or the Cone Penetration Test (CPT) results, the use of field load test results or by the use of pile driving analysis results. An option in the method of analysis to be used could be given.

5. A detailed structural design package could be linked to the expert system for the reinforcement detailing of the pile and the pile cap.

6. The system could be made more attractive by combining it with a package for shallow foundations and improving the graphics.

7. Further expert systems could be developed for the design of other foundation elements such as caissons and also can be used for the design of foundations subjected to dynamic forces.

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